ADDIS ABABA SCIENCE AND TECHNOLOGY UNIVERSITY COLLEGE OF ARCHITECTURE AND CIVIL ENGINEERING SCHOOL OF POST GRADUATE STUDIES

INVESTIGATION ON THE CAUSE OF HIGH WAY FLOODING IN ADDIS ABABA (CASE STUDY ON LEBU ROUNDABOUT - JEMO AND MIKAEL- SEBETA)

MSC. THESIS

BY

ZIYADA BASENA

ADVISOR: NIGUSSIE TEKLIE (PHD)

ADDIS ABABA

FEBRUARY, 2017

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APPROVAL PAGE

This thesis entitled with "*Investigation on the Cause of High Way Flooding in Addis Ababa, Case study on ''Lebu Roundabout - Jemo and Mikael- sebeta*'' has been approved by the following advisor, examiners and head department in the partial fulfillment of Masters of Science in Hydraulic Engineering.

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CERTIFICATION

I, the undersigned, certify that I read and hear by recommend for acceptance by Addis Ababa Science and Technology University a dissertation entitled **"*Investigation on the Cause of High Way Flooding in Addis Ababa, Case study on "Lebu Roundabout - Jemo and Mikael- sebeta*''** in partial fulfillment of the requirement for the degree of Master of Science in Hydraulic Engineering.

____

Dr. Nigussie Teklie

Principal Advisor

DEDICATED

TO

MY FAMILY

DECLARATION AND COPY RIGHT

I, Ziyada Basena, declare that this Thesis is my own original work and that it has not been presented and will not be presented by me to any other University for similar or any other degree award.

Signature: ___________________

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Abbreviation

ERA - Ethiopian Road Authority

AACRA - Addis Ababa City Road Authority

AASHTO - American Association of State Highway and Transportation Officials

FHWA (US) - United State Federal High Way Administration

HEC - Hydrologic Engineering Center

FUPCoB - Federal Urban planning and Coordinating Bureau

MWIE - Ministry of Water Irrigation and Energy

HDS - Hydraulic Design Series

SCS - Soil conservation service

GIS - Geographic Information System

CN - Curve Number

DEM - Digital Elevation Model

DDM - Drainage Design Manual

Abstract

Flood is an excess of water on land that is habitually dry and situation where the inundation is caused by over flow of water in established water course. Its impact is significant in urban area; those impermeable structures decrease amount of water infiltrating, increase surface runoff and result flooding. In Ethiopia more of its urban center receives significant amount of rain fall and Addis Ababa is one of those urban centers with significant amount of rainfall and having many of its roads flooding during the rainy season of the year. This study was intended to investigate the cause of flooding in Addis Ababa, a case study on Lebu Round about to Jemo and Mikael to sebeta. To perform the study primary and secondary data were collected. Using field investigation together with secondary data collected a GIS based hydrologic parameter extraction, peak discharge estimation for streams crossing the road, Hydraulic analysis, comparison between the original design of road and the re designed in this study were the steps followed to investigate the cause of flooding on the study road. Among the structures on the road 35% of structures are not considered, 11% are not consistent with the design and 50% of structures are underestimated in the original design of the road. The result of this study shows that the flood is caused by Hydrologic, Hydraulic, inconsistency in the design versus construction and maintenance delay.

KEY WORDS: Drainage, Peak Flood, Urban Drainage, Urban Flooding

1. INTRODUCTION

Flood is an excess of water on land that's normally dry and is a situation where the inundation is caused by high flow, or overflow of water in an established watercourse, such as a river, stream, or drainage ditch; or pounding of water at or near the point where the rain fell.

The impact of flooding is significant in urban area where it's more costly and difficult to manage. According to (World Bank) 2011, urban flooding is an increasingly important issue. It is a global phenomenon which caused devastation and economic losses. Its impact is driven by a combination of natural and man-made factors i.e. Urbanization and Climate change

Urbanization along with its impermeable structures is the major cause of flooding in urban areas. Urban storm water influences the service life of urban infrastructures. The rainfall intensity and characteristics of catchment area are the major factors for designing urban storm water drainage facilities. These facilities have a paramount advantage to safely dispose the generated floods to ultimate receiving system (Dagnachew , 2011).

Storm drainage design is an integral component in the design of highway and transportation networks. Much of the cost of highway projects is attributable to drainage facilities, including storm drains, highway culverts, bridges, and water quality and quantity control structures. Design of these facilities involves hydrologic analysis and hydraulic analysis to determine the design discharge and of the conveyance capacity of the facility respectively.

Drainage design for highway facilities must strive to maintain compatibility and minimize interference with existing drainage patterns, control flooding of the roadway surface for design flood events and minimize potential environmental impacts from highway related storm water runoff (FHWA, HEC 22, 2013).

Drainage is one of the most important factors to be considered in the road design, construction and maintenance projects. It is generally accepted that road structures work well and last longer to give the desired service. When a road fails, whether it is concrete, asphalt or gravel, inadequate drainage is often a major factor to be considered. Researchers have shown that poor drainage is often the main cause of road damages and problems with long term road serviceability. Though provision of proper road surface drainage systems have such a great importance for the urban road to give the intended use and there by contribute to the overall development of a nation (Getachew K. W., 2015).

Urban storm water drainage facilities are part of the urban infrastructure elements and design of these facilities require due attention. In Ethiopian context, where watersheds of many urban centers receive significant amount of annual rainfall and where rainfall intensity is generally high, control of runoff at source, flood protection and safe disposal of excess water/runoff through proper drainage facilities becomes essential (FUPCoB, 2008)**.**

Inadequate urban storm water drainage problems represent one of the most common sources of compliant from the citizens in many towns of Ethiopia and this problem is getting worse and worse with the ongoing high rate of urbanization (GTZ-IS, 2006).

The situation is more significant in Addis Ababa which is the capital city of Ethiopia, it is highly and rapidly urbanizing city and has many new as well as old roads but high way flooding is a major problem in many of its roads during the rainy seasons of the year. While the roads are flooding, there is high traffic jam staying for hours affecting movement and day to day life of residents, degrading the pavement there by creating economic loss, loss of life and social problems.

Researches were done regarding the cause of flooding in Addis Ababa. The areas of research were to identify Road and urban storm water drainage network integration by Dagnachew (2011). The paper tried to focus on the extent of integration of urban storm water drainage infrastructures in road provision, existing condition of road and urban storm water drainage network, impacts of integration of urban storm water drainage on road performance and related environmental issues and flood prone areas. Whereas, the engineering design aspects of roads and urban storm water drainage net-work, in general, was not part of his study.

Another research by Desalegn (2011) was to investigate storm drainage problems of Addis Ababa. This paper investigated the cause only on pavement drainage but as we know the flood is not only from the pavement but may also be from the streams crossing the road.

Among roads in A.A which are prone to flooding this study focus on *Lebu Round about to Jemo and Mikael to sebeta*. In this road the runoff coming from the nearby mountainous area flows directly to the road in some areas there by creating traffic jam and affecting public movement.

Using the available data, this study tried to identify the cause of this flooding in terms of peak runoff discharge at streams crossing the road, opening requirement of structure to pass this discharge then comparing the original design with the redesigned, original design with implemented on the ground and investigate the cause in terms of capacity and maintenance of existing drainage structures.

1.2. Statement of the Problem

Inadequate urban storm water drainage problems represent one of the most common sources of compliant from the citizens in many towns of Ethiopia and this problem is getting worse and worse with the ongoing high rate of urbanization (GTZ-IS, 2006).

With urbanization, impermeability increases with the increase in impervious surfaces (i.e. residential houses, commercial buildings, paved roads, parking lots), drainage pattern changes, overland flow gets faster, flooding and environmental problems such as land degradation increases. It is a crucial problem facing the existing and future road infrastructure.

In spite of these problems, drainage facilities in most urban centers of the country are nearly absent or at a lower coverage. Planning and design rarely guide construction or provision of such facilities and management (FUPCoB, 2008).

Many roads in A.A are flooded during the rainy season of the year preventing safe traffic flow; facilitate pavement degradation, life as well as economic loss and difficulty in day to day life of peoples. **Annex 1** illustrates road flooding in different parts of Addis Ababa.

The aim of this research is to investigate the cause of flooding in A.A city on *Lebu Round about to Jemo and Mikael to sebeta* road through predominantly flat terrain; in this road the run off coming from the nearby mountainous area flows directly to the road in some areas resulting traffic jam, affecting public movement, degrading the pavement and creating economic loss. The condition of pavement, the status of existing structures, the area affected by the flood and the reconstructed structure are shown in Annex **2**.

From the site visit and site findings the following points were expected to be the cause of flooding at *Lebu Round about to Jemo and Mikael to sebeta* area road.

- Wrong estimation of drainage area size
- Wrong estimation of peak flood
- Land use and land cover change
- Wrong estimation of opening requirement
- Absence of frequent Maintenance (structures are clogged with debris and waste so that they can't pass the design flood)

Thus the cause of flooding is expected to be

- Hydrologic
- Hydraulic
- Maintenance delay causes.

Although these are the expected causes, this study will go deeper into scientific analysis to get into the real causes of the problem.

1.3. Objectives

General objective

- To investigate the cause of high way flooding in Addis Ababa a case study on *Lebu Round about to Jemo and Mikael to sebeta* road.

Specific objectives

- To determine peak discharge of streams crossing the road.
- To determine the opening requirement of structures to pass the discharge
- To compare the original design of the study road with the redesigned.
- To determine the cause of flooding.

1.4. Research Questions

- What are the causes of Addis Ababa's high way flooding?
- What are the remedial measures to be taken to overcome the problem?

1.5. Thesis Layout

The thesis is organized in six chapters: chapter 1 provides brief introduction about Flood and its cause, storm drainage and facilities, the problem that initialize this study, the objective of the study, the outline of the thesis and over all frame work of the research.

Chapters 2 describes the reviewed literature related to previous studies done in Addis Ababa, the concept of urban storm water drainage design, hydrologic and hydraulic analysis methods and selection of methods were described in detail in these chapter.

Chapter 3 Deals with the main parts of the research. Gives detail description of the study area, i.e. location and climate of the study area, method of data collection, data analysis, methodology adopted for hydrologic and hydraulic study were described in detail.

Chapter 4 describes hydrologic and hydraulic study result and discussion of results.

Chapter 5 describes conclusions drawn made based on the result of the study.

Chapter 6 gives recommendation based the result of the study. In addition reference and appendixes are attached at the end.

2. LITERATURE REVIEW

2.1. Previous Studies in Addis Ababa

Researches were done regarding the cause of flooding in Addis Ababa. The areas of research were to identify Road and urban storm water drainage network integration and to investigate storm drainage problems of Addis Ababa.

Dagnachew (2011) in his research paper has assessed the integration of road and urban storm water drainage infrastructure with the help of topographic map and also the condition, pavement type and hierarchy of every road and drain in Addis Ketema Sub-city.

The paper tried to focus on the extent of integration of urban storm water drainage infrastructures in road provision, existing condition of road and urban storm water drainage network, impacts of integration of urban storm water drainage on road performance and related environmental issues and flood prone areas.

Quantitative as well as qualitative data types collected and analyzed with the help of Microsoft Excel, Auto CAD and Arc GIS software and has done descriptive and exploratory type of research. Whereas the engineering design aspects of roads and urban storm water drainage net-work, in general, was not part of his study.

This paper found that the major causes of flooding in the area were blockage of urban storm water drainage lines along with inadequate or poor integration between road and urban storm water drainage infrastructures.

It recommends AACRA should give immediate attention for constructing closed or buried type urban storm water drainage lines in the provision of future road and urban storm water drainage infrastructure and a separate sewerage system. This study strongly recommends improvement in the integration of road and urban storm water drainage infrastructure and integrated solid waste management to prevent over flowing of flood as a result of blockage of drains.

Besides it has noted that some corrective measures should be taken as per the actual design parameters or dimensions with a full consideration of the quantity of run-off generated in the respective catchment.

Another research was done by Desalegne (MSc.) 2011, investigation on storm drainage problems of Addis Ababa. This study deals with investigation of storm drainage problem of Addis Ababa and a possible mitigation measure to overcome the problem and to identify success and short coming of the design and construction of inlet spacing, drainage pipes and drainage at sag point.

On this paper he stated that there are many roads in Addis Ababa facing storm drainage problems, among this roads which are prone to flooding he has selected Ethio china road (Gotera –Wollo sefer), Saris Gotera road (Debrezeit road) and Ring roads.

He has collected primary and secondary data. He categorized the problems in the areas as Construction, management and design problem. With direct field data collection and site visit he tries to investigate management problems and analyzed the construction problem using field survey as well as comparison of design with what is implemented in the ground. Another investigation was on design problems. To do this he redesigns the system using the computation sheet used in American Federal Highway Administration (FHWA) urban drainage design manual and Addis Ababa City Road Authority (AACRA) urban drainage design manual with some modification.

He has performed hydrologic and hydraulic analysis to evaluate the design of storm drainage system. Hydrologic analysis performed with rational method since it is a reasonable formula for computing the peak discharge for small catchment. The evaluation includes inlet spacing, pipe sizing and inlet type selection. He has compared the Values obtained by redesigning with the original design.

He found that the problem in Gotera - Wollo Sefer and Ring road is caused by insufficient drainage operation, over spaced inlet spacing and minimum pipe size is used. The investigation in Saris – Gotera road also shows; the curb inlets are over spaced and constructed with very small opening, the operation system of the drainage is in sufficient and curbs are not constructed according to the design.

These papers investigate the cause only on pavement drainage but as we know the flood is not only from the pavement but may also be from the streams crossing the road.

Elias (2012), on his article quoted the statement of Manaye Ewnetu that Lack of periodic road drainage maintenance is threatening the viability of Addis Ababa city’s road infrastructure. Construction material handling and waste disposal activities like dumping gravel, sand and excavated materials on the side of the public roads which clog the drainage inlet system are resulting in road flooding and causing considerable inconvenience to the public as well as economic losses. Road drainage problems can also be caused by capacity problems i.e. under designed drainage systems.

Improper estimation of the stream discharge coming to the road and wrong estimation of opening requirement to pass this discharge safely under the road might be the cause of flooding.

So that, this study is intended to investigate cause of flooding in A.A city, doing the cause study on *Lebu Round about to Jemo and Mikael to sebeta* road.

2.2. Urban Flooding

Flooding is one of the major natural disasters which disrupt the prosperity, safety and amenity of the residents of human settlements. The term flood encompasses a flow of water over areas which are habitually dry.

Urban floods are floods that happen in a relatively short period of time and can inundate an area with several feet of water. As areas become 'urbanized' or go through the process of urbanization there are increased flood risks that result.

Development of an urban area can have a huge impact on drainage. Rain that has run off impermeable surfaces and travelled via a piped drainage system reaches a river far more rapidly than it did when the land and its drainage was in a natural state, and the result can be flooding and increased pollution (David B. & John W.D, 2004).

Urban floods are more costly and difficult to manage. We can nonetheless examine the functional differences between urban and rural flooding. While rural flooding may affect much larger areas of land and hit the poorest section of the population, the impacts of urban floods are characteristic in that the concentration of population in the urban environment is usually much higher. Therefore damage is more intense and usually more costly (World Bank, 2011).

One of the most important facilities in preserving and improving the urban water environment is an adequate and properly functioning storm water drainage system.

With urbanization, the spatial pattern of flow in the watershed is altered and there is an increase in the hydraulic efficiency of flow through artificial channels, curbing, gutters and storm drainage and collection systems. These factors increase the volume and velocity of runoff and produce larger peak flood discharge from urbanized watersheds than occurred in the pre-urbanized condition.

Many urban drainage systems constructed under one level of urbanization are now operating under a higher level of urbanization and have inadequate capacity. (V.T.Chow, etal, 2004)

2.3. Urban Storm water drainage design

According to (VDOT, 2002) the design of a drainage system must address the needs of the traveling public as well as those of the local community through which it passes. The drainage system for a roadway traversing an urbanized region is more complex than for roadways traversing sparsely settled rural areas. This is often due to:

- The wide roadway sections, flat grades, shallow water courses, absence of side channels
- The potential for more costly property damage which may occur from ponding of water or from flow of water through built-up areas

- The fact that the roadway section must carry traffic, but also act as a channel to convey the water to a disposal point. Unless proper precautions are taken, this flow of water along the roadway could interfere with or possibly halt the passage of highway traffic
- The potential weakening of roadway base and sub grade due to saturation from extensive pounding.

Drainage design covers many disciplines, of which two are hydrology and hydraulics. The determination of the quantity and frequency of runoff, surface and groundwater, is a hydrologic problem. The design of structures with the proper capacity to divert water from the roadway, remove water from the roadway, and pass collected water under the roadway is a hydraulic problem.

For routine design problems, particularly involving small drainage areas, it is impractical and unnecessary to use sophisticated analytical methods that require extensive time and labor (FHWA, HDS 4).

2.3.1. Hydrologic Design

The main purpose of hydrologic study is to compute the peak flood of road-crossing rivers or streams so as to determine the size of drainage structures along the road alignment.

Hydrologic design is the process of assessing the impact of hydrologic events on a water resource system and choosing values for the key variables of the system so that it will perform adequately (V.T.Chow, etal, 2004).

In doing hydrologic analysis for drainage structures there are many variable factors that affect floods. ERA, DDM 2002 states some of the factors that need to be recognized and considered on individual site by site basis:

- Rainfall amount and storm distribution
- Catchment area size, shape and orientation
- Ground cover
- Type of soil

- Slopes of terrain and stream(s)
- Antecedent moisture condition
- Storage potential (overbank, ponds, wetlands, reservoirs, channel, etc.) and
- Catchment area development potential.

Hydrologic analysis requires some parameters which must be extracted from the watershed. Extracting hydrological parameters from the watershed is the first step for doing hydrologic analysis. Previously these parameters were extracted manually from topographic maps, field surveys, or aerial photographs which were a labor-intensive activity. So that there is a need for determining this parameter with the help of computer system capable of assembling, storing, manipulating, and displaying geographically referenced information.

Melat (2015) (MSc.) in her research paper stated that, Cost of most highway projects can be attributed to the design and construction of drainage facilities such as bridges, culverts, storm drains etc. The design of these structures involves hydrologic analyses to determine the design discharge and hydraulic analyses to determine the conveyance capacity. Although most hydrologic and hydraulic calculation procedures are now available as computer programs, which can significantly reduce the mathematical effort involved, substantial effort is still necessary to manipulate the data required for input into these programs.

Traditionally the data generated to support these programs have been extracted manually from maps and cross-sections presented on paper drawings, however by building a digital spatial database of the hydrologic features, a GIS in conjunction with this database, the extraction of data and application of design procedures will become automated and more efficient.

Her paper addresses the method of extracting these hydrological parameters with the help of GIS, Which helps to change the usual method to more developed and accurate method.

2.3.1.1. Geographic Information System (GIS)

GIS is defined as computer systems capable of assembling, storing, manipulating, and displaying geographically referenced information (U.S.Geographic Survey, 1998)

Originally developed as a tool for cartographers, GIS has increasingly been used in engineering design and analysis, especially in the fields of water quality, hydrology, and hydraulics. GIS provides a setting on which to overlay data layers and perform spatial queries, thus creating new data. The results can be digitally mapped and tabulated, facilitating efficient analysis and decision making. Structurally, GIS consists of a computer environment that joins graphical elements (points, lines, polygons) with associated tabular attribute descriptions. This characteristic sets GIS apart from both computer-aided design software (geographic representation) and databases (tabular descriptive data). For example, in a GIS view of a group of rivers, the graphical elements would represent the location and shape of the rivers, whereas the attributes might describe the stream name, length, and flow rate. This one-to-one relationship between each feature and its associated attributes makes the GIS environment unique.

In a relatively short time, GIS has gained widespread use in a variety of engineering applications. Originally envisioned (and used) as a geographic mapper with integrated spatial database, GIS is increasingly being used in modeling applications, where geographic data can be readily accessed, processed, and displayed. Historically, GIS has been implemented primarily by large entities, such as federal, state, and local government agencies, predominantly for mapping and management of spatial data. However, there is increasing interest in the potential application of GIS in engineering design and analysis, especially in hydrology and hydraulics. (David, etal, 1998)

Geographic Information Systems (GIS) link land cover data to topographic data and to other information concerning processes and properties related to geographic location. Non topographic information can include description of soils, land use, ground cover, and ground water condition, as well as man-made systems and their characteristics on or below the land surface.

While maps have been the most common historical form of representing topography, the advert of digital maps in GIS provides an alternative method of storing and retrieving this information. The amount of digital data required to accurately describe the topography of even small geographic regions make GIS a memory intensive and computationally complicated system.

The characteristic that differentiates a GIS from general computer mapping or drawing systems is the link to the information Data base. Once the data base is constructed, correlations between different pieces of information can be examined easily through computer-generated overlay maps. For hydrologic modeling purposes, there is generally an extra step off generating hydrologic parameters that are dependent on data-base information. This hydrology GIS link is a significant complicating factor, because it involves complex empirical or physics-based relations. (Arlen D.F. and Bruce A.De.Vantier, 1993)

GIS uses model for building hydrologic information systems to synthesize geospatial and temporal water resources data that support hydrologic modeling and analysis, which is called Arc Hydro model.

2.3.1.1.1. Arc Hydro Model

Arc Hydro is a model developed as an Add-on to Arc GIs software. It is used to extract topologic variables from a digital elevation model raster (DEM) for building geometric networks for hydrologic analysis.

This geospatial and temporal model supports hydrologic simulation models categories to divide water resources elements, such as network, drainage, channel, hydrograph, and time series. The Arc Hydro tools (part of the Arc Hydro Add on toolbar) generate several datasets that collectively describe the drainage pattern of a catchment. Most watershed managers use this facility more than other advanced hydrologic analysis for their watershed management requirement. At the outset, a raster analysis is performed to generate data on flow direction, flow accumulation, stream definition, stream segmentation, and finally watershed delineation. Next, these data are used to develop a vector representation of catchments and drainage lines and in the end to help in constructing a geometric network. (Melat kaleab, 2015)

Arc Hydro is an Arc GIS-based system geared to support water resources applications. It consists of two key components, Arc Hydro Data Model and Arc Hydro Tools

These two components, together with the generic programming framework, provide a basic database design and a set of tools that facilitate the analyses often performed in the water resources area. Arc Hydro is intended to provide the initial functionality that can then be expanded by adding to its database structures and functions required by a specific task or application.

The Arc Hydro tools operate in the Arc GIS environment. Some of the functions require the Spatial Analyst extension. The majority of the tools are accessed through the Arc Hydro Tools toolbar, where they are grouped by functionality into six menus and nine tools. Additional tools have been developed in the geo-processing environment and are available in the Arc Hydro Tools toolbox that can be used both in Arc Map and in Arc Catalog.

The Arc Hydro tools have two key purposes. The first purpose is to manipulate (assign) key attributes in the Arc Hydro data model. These attributes form the basis for further analyses. They include the key identifiers (such as Hydro ID, Drain ID, Next Down ID, etc.) and the measure attributes (such as Length Down). The second purpose for the tools is to provide some core functionality often used in water resources applications. This includes DEM-based watershed delineation, network generation, and attribute-based tracing.

The functionality of Arc Hydro tools is expected to grow over time. They have been implemented in a way that allows easy addition to their functionality, either internally (by adding additional code) or externally, by providing additional functionality through the use of key Arc Hydro data structures. (ESRI, 2011)

Catchment characterization and modeling activity is highly dependent on the existing digital elevation models with adequate spatial resolution and covering the whole area of interest is a basic requirement to achieve the goal of catchment characterization and modeling activity. The quality of DEM is very important and great attention is needed to avoid errors that are introduced by the terrain model itself. Since the propagation of errors play a significant role in GIS analysis.

Catchment models are in general designed to meet one of two primary objectives. The first is to gain a better understanding of the hydrologic behaviors of a catchment and of how changes in the catchment may affect these behaviors. The second objective of catchment modeling is the generation of synthetic hydrologic data for facility design like water resources planning, flood protection, mitigation of contamination, or licensing of abstraction or for forecasting. They are also providing valuable information for studying the potential impacts of changes in land use or climate (Melat, 2015).

In Ethiopia, stream flow measurements for determining a flood frequency relationship at or near a site are usually unavailable. In such cases, it is an accepted practice to estimate peak runoff rates and hydrographs using statistical or empirical methods. The peak discharge is adequate for design of conveyance systems such as storm drains, open channels, culverts, and bridges.

Each method has a range of application and limitations, which the engineer should clearly understand prior to using them. Basin size, hydrologic and geographic region, dominant precipitation type, elevation, and level of development are all important factors. The engineer must ensure that the selected hydrologic method is appropriate for the basin conditions and that sufficient data is available to perform the required calculations (ERA, 2013).

Some of the hydrologic methods approved by ERA:

- **Rational method** shall be used only for catchment areas less than 80 hectares (0.8 km^2);
- **SCS and other unit hydrograph methods** for catchment areas greater than 80 hectares;
- **Catchment area regression equations** shall be used for all routine designs at sites where applicable;
- **Gumble or Log Pearson III analyses** shall preferably be used for all routine designs provided there is at least 10 years of continuous or synthesized record for 10-year discharge estimates and 25 years for 100-year discharge estimates;
- Suitable computer programs such as **HYDRAIN**'s **HYDRO**, **HEC 1**, and **TR-20** may be used to facilitate tedious hydrologic calculations.

Of these possible hydrologic methods ERA noted that only Rational and SCS methods are applicable to the whole country (Ethiopia). Although Regression equation and derivations from stream gauging (Gumbel, Log Pearson) are often preferred they rely on unavailable data.

2.3.1.2. Rational Method

It is one of most commonly used equations for calculation of peak discharge for drainage area less than or equal to 80ha. The peak discharge for the rational method occurs when the entire watershed is contributing.

AASHTO, 2014 also states that with few exceptions, runoff estimates for drainage design are made by using rational methods. In order to make use of these methods, information on the intensity, duration, and frequency of rainfall for the locality of the project must be established.

V.T. Chow and his friends on their applied hydrology book stated that the idea behind the rational method is:

If a rainfall of intensity $\boldsymbol{i}$ begins instantaneously and continues indefinitely, the rate of runoff will increase until the time of concentration $\boldsymbol{t_c}$, when all of the watershed is contributing to flow at the outlet.

The product of rainfall intensity $\boldsymbol{i}$ and watershed area $\boldsymbol{A}$ is the inflow rate for the system and the ratio of this rate to the rate of peak discharge $\mathbf{Q_p}$ (which occurs at time $\boldsymbol{t_c}$) is termed the runoff coefficient **C** ($0 < C < 1$). This is expressed in the rational formula:

$$\boldsymbol{Q_p = 0.278CiA} \qquad (2\text{-}1)$$

Where

Qp = the peak discharge for the required return period (m^3/s)

C = the runoff coefficient

I = the rainfall intensity for a required return period of duration equal to critical storm duration (mm/hr)

A = the drainage watershed area (km^2)

In urban areas, the drainage area usually consists of subareas or sub catchments of different surface characteristics. As a result, a composite analysis is required that must account for the various surface characteristics. The areas of the sub catchments are denoted by A; and the runoff coefficients of each sub catchment are denoted by C; the peak runoff is then computed using the following form of the rational formula:

$$\boldsymbol{Q = i \sum_{j=1}^{m} C_j A_j} \qquad (2\text{-}2)$$

Where, m is the number of sub catchments drained by a sewer. (V.T. Chow, *etal* 2004)

2.3.1.2.1. Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate in mm/hr for duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the catchment area, the rainfall intensity can be determined from Rainfall-Intensity-Duration curves. (ERA , 2002)

For drainage areas in Ethiopia, the rainfall intensity can be computed at any required time using the 24hr rainfall depth, which is known as a rainfall intensity-duration-frequency (IDF) relationship.

$$\boldsymbol{R_{Rt} = \frac{t(b+24)^n}{24(b+t)^n}} \qquad (2\text{-}3)$$

Where:

R_{Rt} = Rainfall depth Ratio R_t: R_{24}

R_t = Rainfall depth in a given duration *t*

R_{24}= 24 hr rainfall depth

b and n = coefficients b=0.3 and n= (0.78-1.09).

2.3.1.2.2. Runoff coefficient

The runoff coefficient (C) in the rational formula is the ratio of the rate of runoff to the rate of rainfall at an average intensity (i) when all the drainage area is contributing. The runoff coefficient is tabulated as a function of land use conditions; however, the coefficient is also a function of slope, intensity of rainfall, infiltration and other abstractions. (FHWA, HDS4)

According to FHWA, HEC 12, the runoff coefficient, C, characterizes antecedent precipitation, soil moisture, infiltration, detention, ground slope, ground cover, evaporation, shape of the watershed and other variables. For relatively small watersheds such as those dealt with in the surface drainage of highway pavements, adjustments are probably unwarranted. Average values for various surface types, which are assumed not to vary during the storm, are commonly used. Where drainage areas are composed of parts having different runoff characteristics, a weighted coefficient for the total drainage area is computed by dividing the summation of the products of the area of the parts and their coefficients by the total area, i.e.,

$$\frac{C1A1+C2A2+\cdots+CnAn}{An} \qquad (2\text{-}4)$$

Recommended runoff coefficients for urban area are shown below.

Description of Area	Runoff Coefficients
Business: Downtown areas	0.70-0.95
Neighborhood areas	0.50-0.70
Residential: Single-family areas	0.30-0.50
Residential:Multi units, detached	0.40-0.60
Residential: Multi units, attached	0.60-0.75
Suburban	0.25-0.40
Residential (0.5 hectare lots or more)	0.30-0.45
Apartment dwelling areas	0.50-0.70
Industrial: Light areas	0.50-0.80
Industrial: Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Playgrounds	0.20-0.40
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30

Table 2-1.values of runoff coefficient, C (FHWA, HEC 19, 1984)

2.3.1.3. Time of concentration

Time of concentration is the time it takes for runoff to travel from the hydraulically most distant point in the watershed to the point of reference downstream. An assumption implicit to the Rational Method is that the peak runoff rate occurs when the rainfall duration lasts as long as or longer than the time of concentration. Therefore, the time of concentration for the drainage area must be estimated in order to select the appropriate value of rainfall intensity for use in the equation. (FHWA, 1984)

For urban areas, values of *tc* are normally calculated as length divided by velocity determined by hydraulic formulas. For rural drainage basins, tc is generally estimated by means of an empirical formula such as Kirpich's equation (Yelma, 2005).

As indicated in ERA drainage design manual (2013), three common errors should be avoided when calculating Tc. First, application of simplified general equations such as Kirpich for determining Tc can result in too short a time of concentration, particularly when the average basin slope varies significantly from the mean channel slope as in steep mountainous areas. Neglecting the overland flow time can also dramatically shorten the time of concentration thus increasing the design peak runoff.

2.3.1.3.1. Methods for estimating time of concentration

a. Time of Concentration for Overland Flow

Overland flow is the type of flow that occurs in small, flat or in upper reaches of catchments, where there is no clearly defined watercourse. Run-off, then, is in the form of thin layers of water flowing slowly over the fairly uneven ground surface. The kerby formula is recommended for the calculation of Tc in this case. It is only applicable to parts here the slope is fairly even.

$$\boldsymbol{T_c = 0.604\left(\frac{rL}{S^{0.5}}\right)^{0.467}} \qquad \textit{(2-4)}$$

Where:

T_c= time of concentration (hours)

r = roughness coefficient

L=hydraulic length of catchment, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km)

$$\boldsymbol{S = \frac{H}{1000L}} \qquad (2\text{-}6)$$

S=Slope of the catchment

H =height of most remote point above outlet of catchment (m)

b. Time of Concentration for Defined Watercourses

In a defined watercourse, channel flow occurs. The recommended empirical formula for calculating the time of concentration in natural channels was developed by the US Soil Conservation Service.

$$\boldsymbol{T_c = \left(\frac{0.87L^2}{1000S_{av}}\right)^{0.385}} \qquad (2\text{-}5)$$

Where:

T_c = time of concentration (hours).

L = hydraulic length of catchments measured along flow path from the catchment boundary to the point where the flood needs to be determined (km).

S_{av}= average slope (m/m).

The formula for determining the slope according to the 1085-slope methods reads:

$$\boldsymbol{S_{av} = \frac{H_{0.85L} - H_{0.10L}}{1000*0.75L} \text{ or } S_{av} = \frac{H}{1000*0.75L}} \qquad (2\text{-}8)$$

Where:

S_{av} =average slope (m/m)

$H_{0.10L}$ =elevation height at 10% of the length of the watercourse (m)

$H_{0.805L}$ = elevation height at 85% of the length of the watercourse (m)

L = length of watercourse (km)

$H = H_{0.805L} - H_{0.10L}$ (m)

c. Time of Concentration for Urban Areas

In urban areas, the time of concentration should be determined, where applicable, by means of the flow velocities according to the Chezy or Manning's equation for uniform flow through representative cross-sections with representative slopes.

In road drainage, the volume of water that runs off as a result of a storm of less than 15 minute duration is usually not large; much of this runoff is absorbed in filling of watercourses. Times of concentration of less than 15 mins are thus generally not significant.

It is sound practice to calculate the average flow velocity ($v = L/Tc$) after determining Tc in order to ensure that it falls within realistic times. Typical value of the flow velocity ranges from 0.1 to 4m/s, depending on the natural conditions.

d. Watershed lag method

The SCS method for watershed lag was developed by Mockus in 1961. It spans a broad set of conditions ranging from heavily forested watersheds with steep channels and a high percent of runoff resulting from subsurface flow, to meadows providing a high retardance to surface runoff, to smooth land surfaces and large paved areas.

$$\boldsymbol{L = \frac{l^{0.8}(S+1)^{0.7}}{1900Y^{0.5}}} \qquad (2\text{-}6)$$

$$\boldsymbol{L = 0.6Tc} \qquad (2\text{-}7)$$

$$\boldsymbol{Tc = \frac{l^{0.8}(S+1)^{0.7}}{1140Y^{0.5}}} \qquad (2\text{-}8)$$

Where:

L = lag, hr

Tc = time of concentration, hr

l = flow length, ft

Y = average watershed land slope, %

S = maximum potential retention, in $S = \frac{1000}{CN} - 10$

e. The Kirpich Method

This method is ideal for natural basins with well-defined channels. This method is still in relatively heavy use. For channel-flow component of runoff, the Kirpich equation is:

$$\boldsymbol{tc = \frac{KL^{0.77}}{S^{0.385}}} \qquad (2\text{-}9)$$

Where:

- t_{ch} = the time of concentration, in minutes
- K = a units conversion coefficient, in which K = 0.0078 for traditional units and K = 0.0195 for SI units
- L = the channel flow length, in feet or meters as dictated by K
- S = the dimensionless main-channel slope

If it is determined that a low slope condition or a transitional slope condition exists, the user should consider using an adjusted slope in calculating the time of concentration.

2.3.1.4. Drainage Area

The catchment area and land-use are the most important for good prediction of storm water runoff. The boundaries of the complete catchment to be drained can be defined with a catchment modeling tool which is Arc hydro modeling tool.

Catchment models are in general designed to meet one of two primary objectives. The first is to gain a better understanding of the hydrologic behaviors of a catchment and of how changes in the catchment may affect these behaviors. The second objective of catchment modeling is the generation of synthetic hydrologic data for facility design like water resources planning, flood protection, mitigation of contamination, or licensing of abstraction or for forecasting.

They are also providing valuable information for studying the potential impacts of changes in land use or climate (Melat kaleab, 2015).

2.3.1.5. Soil conservation service (SCS) method

Techniques developed by the U. S. Soil Conservation Service for calculating rates of runoff require the same basic data as the Rational Method: catchment area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more sophisticated in that it considers also the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. With the SCS method, the direct runoff can be calculated for any storm, either real or fabricated, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess. (ERA, DDM, 2002).

The method can be used for drainage areas which include areas outside the highway pavement, as for roadside ditches and drainage systems which combine highway pavement drainage with other drainage.

Application of the method requires identification of hydrologic soil groups, watershed area, percent impervious, and overall slope. (HEC 12)

2.3.1.5.1. Rainfall

The SCS method is based on a 24-hour storm event which has a Type II time distribution. The Type II storm distribution is a ’typical" time distribution which the SCS has prepared from rainfall records. It is applicable for interior rather than the coastal regions and should be appropriate for Ethiopia. The Type II rainfall distribution will usually give a higher runoff than a Type I distribution. To use this distribution it is necessary for the user to obtain

1) the 24-hour rainfall value for the frequency of the design storm desired, and then
2) Multiply this value by 24 to obtain the total 24-hour storm volume in millimeters. (ERA, DDM, 2002)

2.3.1.5.2. Rain fall runoff equation

The SCS runoff equation is a method of estimating direct runoff from 24 hour storm rainfall. The method was developed based on 24-hr rainfall runoff data in USA. In its derivation it is assumed that no runoff occurs until rainfall equals an initial abstraction (that is losses before runoff begins) *Ia*, and also satisfies cumulative infiltration *F* (the actual retention before runoff begins) or water retained in the drainage basin, excluding *Ia*. The potential retention (the potential retention before runoff begins) *S* is the value that *(F + Ia)* would reach in a very long storm. (Yelma Seleshi(Dr.), 2005)

The Soil Conservation Service (1972) developed a method for computing abstractions from storm rainfall. For the storm as a whole, the depth of excess precipitation or direct runoff P_e is always less than or equal to the depth of precipitation *P*: likewise, after runoff begins, the additional depth of water retained in the watershed, F_e is less than or equal to some potential maximum retention S

There is some amount of rain fall I_a (initial abstraction before Ponding) for which no runoff will occur, so the potential runoff is $P - I_a$. The hypothesis of the SCS method is that the ratios of the two actual to the two potential quantities are equal. That is,

$$\frac{Fa}{S} = \frac{Pe}{P-Ia} \qquad (2\text{-}10)$$

From continuity principle:

$$P = Pe + Ia + Fa \qquad (2\text{-}11)$$

Combining the two equations to solve for P_e gives;

$$P_e = \frac{(P-I_a)^2}{P-I_a+S} \qquad (2\text{-}12)$$

Where: P_e = Depth of direct runoff, mm (in)

P = Depth of 24-hour precipitation, mm (in),

S = Retention, mm (in)

This is the basic equation, for computing the depth of excess rainfall or direct runoff from a storm by the SCS method.

Studying results from many small experimental watersheds an empirical relation was developed.

$$\boldsymbol{Ia = 0.2S} \qquad (2\text{-}13)$$

Substituting equation 9 into equation 8 will result

$$\boldsymbol{P_e = \frac{(P-0.2S)^2}{P-0.8S}} \qquad (2\text{-}14)$$

Plotting the data for P and P_e from many watersheds the SCS found curves. To standardize these curves a dimensionless curve number CN is defined such that $0 \leq CN \leq 100$. For impervious and water surfaces CN = 100: for natural surfaces CN < 100. (V.T. Chow, *etal.)*

Empirical studies found that S is related to soil type, land cover, and the antecedent moisture condition of the basin. These are represented by the runoff curve number, CN, which is used to estimate S with the following equation:

$$\boldsymbol{S = \frac{1000}{CN} - 10} \qquad (2\text{-}15)$$

Where, S is in inches. (HEC 22, UDDM)

2.3.1.5.3. Runoff factors

Runoff is rainfall excess or effective rainfall - the amount by which rainfall exceeds the capability of the land to infiltrate or otherwise retain the rainwater. The principal physical catchment area characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope.

Land use is the catchment area cover, and it includes both agricultural and nonagricultural uses. Items such as type of vegetation, water surfaces, roads, roofs, etc. are all part of the land use.

Land treatment applies mainly to agricultural land use, and it includes mechanical practices such as contouring or terracing and management practices such as rotation of crops.

Soil type influences the relationship between rainfall and runoff by affecting the rate of infiltration. The SCS has divided soils into four hydrologic soil groups based on infiltration rates (Groups A, B, C, and D).

The SCS uses a combination of soil conditions and land-use (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher is the runoff potential. (ERA, DDM, 2002)

Curve numbers have been tabulated by the Soil Conservation Service on the basis of soil type and land use. Four soil groups are defined:

Group A: Deep sand, deep loess, aggregated silts

Group B: Shallow loess, sandy loam

Group C; Clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay

Group D: Soils that swell significantly when wet, heavy plastic clay and certain saline soils. For a watershed made up of several soil types and land uses, a composite CN can be calculated. (V.T.Chow, *etal*)

Runoff curve numbers vary with the antecedent soil moisture conditions, defined as the amount of rainfall occurring in a selected period preceding a given storm. In general, the greater the antecedent rainfall, the more direct runoff there is from a given storm. A five-day period is used as the minimum for estimating antecedent moisture conditions. Antecedent soil moisture conditions also vary during a storm; heavy rain falling on a dry soil can change the soil moisture condition from dry to average to wet during the storm period. (ERA, DDM, 2002)The table below gives CN values for urban area for normal (average) antecedent moisture content.

Cover description	Curve numbers for hydrologic soil groups				
Cover type and Hydrologic condition	Average % impervious area[2]	A	B	C	D
Open space (lawns, parks, cemeteries, etc.)[3]					
Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50 % to 75%)		49	69	79	84
Good condition (grass cover >75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Desert urban areas:					
Natural desert cover		63	77	85	88
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
0.05 hectare or less	65	77	85	90	92
0.1 hectare	38	61	75	83	87
0.135 hectare	30	57	72	81	86
0.2 hectare	25	54	70	80	85
0.4 hectare	20	51	68	79	84
0.8 hectare	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94

[1] Average runoff condition, and I_a=0.2S

Table 2-2. CN values of urban area for normal (average) antecedent moisture content. (ERA DDM, 2013)

Total 5 day antecedent rain fall		
AMC group	Dormant season	Growing season
I	Less than 0.5	Less than 1.4
II	0.5-1.1	1.4-2.1
III	Over 1.1	Over 2.1

Table 2-3. Classification of antecedent moisture class for the SCS (ERA, DDM, 2002)

As V.T. Chow and his friend stated, curve obtained (***Appendix 13***) should be applied for normal antecedent moisture condition (AMC II). For dry conditions (AMC I) or wet conditions (AMC III) equivalent curve numbers can be computed by:

$$\boldsymbol{CN(I) = \frac{4.2CN(II)}{10-0.058CN(II)}} \qquad (2\text{-}16)$$

$$\boldsymbol{CN(III) = \frac{23CN(II)}{10-0.13CN(II)}} \qquad (2\text{-}17)$$

Care shall be taken in the selection of curve numbers (CN’s). Use a representative average curve number, CN, for the catchment area. Selection of overly conservative CN’s will result in the estimation of excessively high runoff and consequently excessively costly drainage structures. Selection of conservatively high values for all runoff variables results in compounding the runoff estimation. It is better to use average values and design for a longer storm frequency.

ERA, DDM, 2002 recommends antecedent moisture conditions (AMC) in Ethiopia, to use dry for Region D1, wet for Region B1, and average AMC for all other regions and for portion of Region A2 in the vicinity of Bahir Dar to be treated as wet. When wet AMC is used, it is unlikely that the vegetation density will also be poor to sparse.

2.3.1.6. **Design Frequency**

A design frequency shall be selected to match the structure’s cost, amount of traffic, potential flood hazard to property, expected level of service, political considerations, and budgetary constraints, and magnitude and risk associated with damages from larger flood events. In case of long highway routes, having no practical detour, and where many sites are subject to independent flood events, it may be necessary to increase the design frequency at each site to avoid frequent route interruptions from floods. In selecting a design frequency, potential upstream land use that could reasonably occur over the anticipated life of the drainage facility shall be considered. (ERA, DDM, 2001).

Hydrologic analysis should include the determination of several design flood frequencies for use in the hydraulic design. These frequencies are used to size different drainage structures to allow for an optimum design, that considers both risk of damage and construction cost. Consideration shall be given to what frequency flood was used to design other structures along a highway corridor.

Since it is not economically feasible to design a structure for the maximum runoff a catchment area is capable of producing, a design frequency must be established. The frequency with which a given flood can be expected to occur is the reciprocal of the probability or chance that the flood will be equaled or exceeded in a given year. If a flood has a 20 percent chance of being equaled or exceeded each year, over a long period of time, the flood will be equaled or exceeded on an average of once every five years. This is called return period or recurrence interval (RI). Thus the exceedence probability equals 100/RI.

Cross Drainage facility shall be designed to accommodate a discharge with a given return period(s) for the following circumstances. The design shall be such that the backwater (the headwater) caused by the structure for the design storm does not:

• Increase the flood hazard significantly for property;
• Overtop the highway; or
• Exceed a certain depth on the highway embankment.

Based on these design criteria, a design involving roadway overtopping of short duration for floods larger than the design event is acceptable practice. Usually, if overtopping is allowed, the structure may be designed to accommodate a flood of some lower frequency without overtopping.

Storm Drains: shall be designed to accommodate a discharge with a given return period(s) for the following circumstances. The design shall be such that the storm runoff does not:

• Increase the flood hazard significantly for property;
• Encroach on to the street or highway so as to cause a significant traffic hazard; or
• Limit traffic, emerging vehicle, or pedestrian movement to an unreasonable extent.

Based on these design criteria, a design involving a street or road inundation of short duration for floods larger than the design event is an acceptable practice. (ERA,DDM, 2002)

A complete drainage system is composed of major and minor drainage systems. Minor drainage systems consist of components historically considered being part of drainage system. These components include curbs, gutters, ditches, inlets, pipes and other conduits. The minor system is designed for a flood of 10 year ARI.

The major system provides the overland relief for storm water exceeding the capacity of minor systems. This relief is necessitated during less frequent storm such as 25, 50 and 100 year events. The major system is composed of pathways provided for runoff to flow to natural or man-made channels. (AACRA,DDM, 2004)

2.3.1.7. SCS hydrograph Peak discharge determination

The peak discharge in the SCS method is derived from the triangular approximation to the hydrograph shown in Figure 1, resulting from rainfall excess of duration D

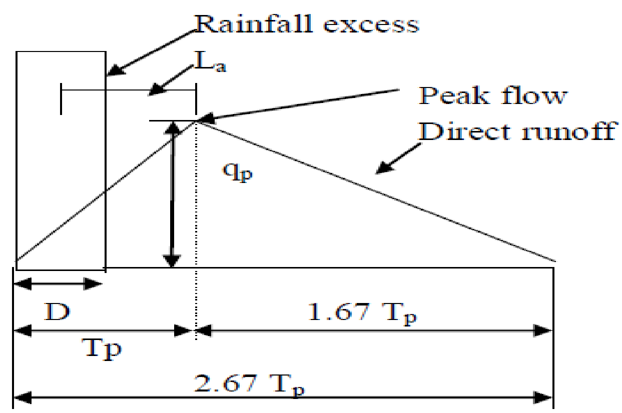


Figure1. SCS triangular hydrograph

The lag La of the peak flow, time from the centroid of rainfall excess to the peak of the hydrograph, is assumed to be 0.6tc. Then the time of rise Tp to the peak of the hydrograph is

$$\boldsymbol{T_p = 0.5D + 0.6T_c} \qquad (2\text{-}18)$$

In hydrograph analysis, time of concentration is the time from the end of excess rainfall to the point on the falling limb of the dimensionless unit hydrograph (point of inflection) where the recession curve begins.

Various researchers (Mockus 1957; Simas 1996) found that for average natural watershed conditions and an approximately uniform distribution of runoff:

$$L = 0.6Tc \qquad (2\text{-}19)$$

Where:

L = lag, hr

Tc = time of concentration, hr

When runoff is not uniformly distributed, the watershed can be subdivided into areas with nearly uniform flow so that equation 13 can be applied to each of the subareas.

The base length of the hydrograph is assumed to be 2.67T_p. Then from a triangular hydrograph assumption (excess rainfall depth = runoff depth) the peak discharge can then be estimated from

$$Q_p = \frac{0.208Ap_e}{0.5{T_c}^{05}+0.6T_c} \qquad (2\text{-}20)$$

Where:
Qp = peak discharge (m^3/s)

Pe = the excess rainfall depth (mm) determined from Equation (2-24)

A = watershed area (km^2)

T_c= time of concentration (hr)

D = duration of excess rainfall (hr)

The depth of runoff resulting from a required return period rainfall depth of duration corresponding to the time of concentration Tc is estimated by

$$P_e = \frac{(P-0.2S\,)^2}{P-0.8S} \qquad \textbf{(2-21)}$$

Where:

Pe= depth of direct runoff equal to depth of excess rainfall (mm)

S = the potential retention (mm)

P = design rainfall amount of duration tc corresponding to T years return period (mm)

S in (mm) is estimated using, $\boldsymbol{S = 254(\frac{100}{CN} - 1)}$ *(2-22)*

The explicit consideration of the various factors that are thought to affect flood runoff makes the method (SCS) attractive. Designers however may have uncertainties in choosing the CN and in determining the method for Tc. It is found that assumed antecedent moisture condition had major effect and that results were better for bare soil or sparse vegetation than for dense vegetation. (Dr. Yelma Seleshi, 2005)

2.3.1.8. Soil Conservation Service (SCS) Graphical Peak Discharge Determination

Using SCS relation of direct runoff, curve number and precipitation (direct rainfall) curve *(Ref. Appendix 13)* peak discharge can be estimated by;

$$\boldsymbol{Q_{p_n} = Q_u x\, A\, x\, P_{en}} \qquad (2\text{-}23)$$

Where

Qp_n=Peak discharge for *n* year return period

Q_u =Unit Peak Discharge $m^3/s/Km^2/mm$ determined from Tc vs. Ia/P curve Ref. *Appendix 14*

P_{en}= Depth of direct runoff, mm for n year return period

The computation of effective precipitation, P_{en}, forming flood and the CN number are computed as in the above.

The unit peak discharge is obtained from the following equation, which requires the time of concentration (tc) in hours and the initial abstraction rainfall (Ia/p) ration as input:

$$\boldsymbol{Q_u = \propto * 10^{Co + C1 logtc + C2 (logtc)^2}} \qquad (2\text{-}24)$$

Where Co, C1 and C2 = regression coefficients given in *Appendix 16* for various Ia/p ratios:

$\propto$ = unit conversion factor equal to 0.000431 in SI unit.

2.3.1.8.1. Ia/P Parameter

Ia/P is a parameter that is necessary to estimate peak discharge rates. Ia denotes the initial abstraction and P is the 24 hour rainfall depth for a selected return period. For a given 24 hour rainfall distribution Ia/P represents the fraction of rainfall that must occur before runoff begins.

2.3.2. Hydraulic Design

Once the design discharge coming from catchment area has been calculated by one of appropriate method the next step is to determine the opening sizes of the drainage structures required to pass this discharge through the structure.

2.3.2.1. Manning's Formula of Hydraulic Analysis

This method deploys the hydraulic characteristics of the stream influencing the maximum discharge, such as velocity of flow, slope of the stream, cross sectional area of the stream and shape and roughness of the stream. It is most widely used equation for uniform open channel flow calculations. This method is used for the design flood levels at crossing sites after the design discharges have been estimated by the one of appropriate hydrological methods.

Accordingly, the following Manning's equation can be used for high-water computations in the hydraulic design of drainage structures:

$$\boldsymbol{Q = K\frac{1}{n} * R^{\frac{2}{3}} * S^{\frac{1}{2}}} \qquad \textit{(2-25)}$$

Where:

Q	=	Discharge in [m3/sec]
R	=	Hydraulic mean depth [m] = A/P
A	=	Cross-sectional flow area [m^2]
P	=	Wetted perimeter [m]
S	=	Longitudinal bed slope [%]
n	=	Manning's roughness coefficient
K	=	constant, 1 for SI and 1.49

Uniform open channel flow which is required for use of the Manning equation occurs for a constant flow rate of water through a channel with constant slope, size, shape, and roughness. This situation exists when there is partially full flow.

Uniform partially full pipe flow occurs for a constant flow rate of water through a pipe of constant diameter, surface roughness and slope. Under these conditions the water will flow at a constant depth. S is actually the slope of the hydraulic grade line. For uniform flow, the depth of flow is constant, so the slope of the hydraulic grade line is the same as the slope of the liquid surface and the same as the channel bottom slope. The channel bottom slope is typically used for S in the Manning equation.

The common flow condition in urban drainage pipes is part-full flow. The presence of the free surface must be taken into account in hydraulic computations.

In uniform steady gravity flow, equilibrium exists along a part-full pipe or channel. The energy consumed by friction between the liquid and the pipe wall is in balance with the fall along the pipe length. If pipe slope could be increased for the same flow-rate, additional energy would be available to the flow, resulting in higher velocity and lower depth. The equilibrium depth is referred to as the ***normal depth*.**

Since depth of flow and velocity are constant when conditions are uniform, and pressure at the surface is atmospheric, the EGL and HGL are parallel to the bed, and the HGL coincides with the water surface. (David Butler, 2004)

2.3.2.2. Normal Depth Calculation

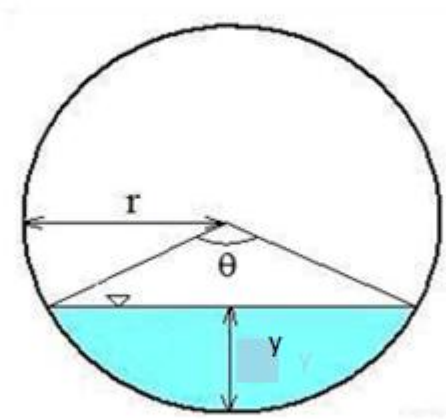
For a constant flow rate through a channel with constant bottom slope, cross-sectional shape and size, and Manning roughness coefficient, the depth of flow will be constant at a depth called the normal depth. The procedure for determining the normal depth is the same for gravity flow through partially full pipes as it is for open channel flow with cross-sectional shapes like rectangular or trapezoidal.

$$\boldsymbol{A\left({R_h}^{\frac{2}{3}}\right)/n = Q/K\left(S^{1/2}\right)} \qquad \textit{(2-26)}$$

For flow in a channel with specified Q, n, and S, the right hand side of the equation has a constant value that can be calculated. The left hand side of the equation can be written as a function of the depth of flow, y, for a specified channel shape and size. An iterative solution is typically required to find the value of **y** that makes the two sides of the equation equal.

Calculation of the flow rate and average velocity, or determination of normal depth at a given flow rate, for partially full pipe flow can be carried out with the Manning equation in a manner similar to such calculations for traditional open channel cross-sections, like rectangular or trapezoidal. (Harlan H.Bengston, 2010)

The equations for calculating the cross-sectional area of flow, A, the wetted perimeter, P, and the hydraulic radius, R_h, for partially full flow are shown below.



$$h = y \qquad r = D/2$$

$$\theta = 2arccos\left(\frac{r-y}{r}\right)$$

$$A = \frac{r^2(\theta - sin\theta)}{2}$$

$$P = r\theta$$

$$R_h = \frac{A}{P}$$

In part-full pipes, maximum flow velocity and flow-rate do not occur when the pipe is running full; they occur when it is slightly less than full. This is because the circular shape affects the relative magnitudes of the flow area and the wetted perimeter (which determines the magnitude of the frictional resistance) (David,2004).

3. MATERIALS AND METHODS

3.1. Study Area

3.1.1. Location

The study is conducted in Addis Ababa, capital city of Ethiopia, where significant storm drainage problem exists. The city lies at an altitude of 2,300 m and located 9°1′48″N and 38°44′24″E coordinates.

The city lies at the foot of Mount Entoto and forms part of the watershed for the Awash. From its lowest point, around Bole International Airport, at 2,326 m above sea level in the southern periphery, the city rises to over 3,000 meters in the Entoto Mountains to the North.

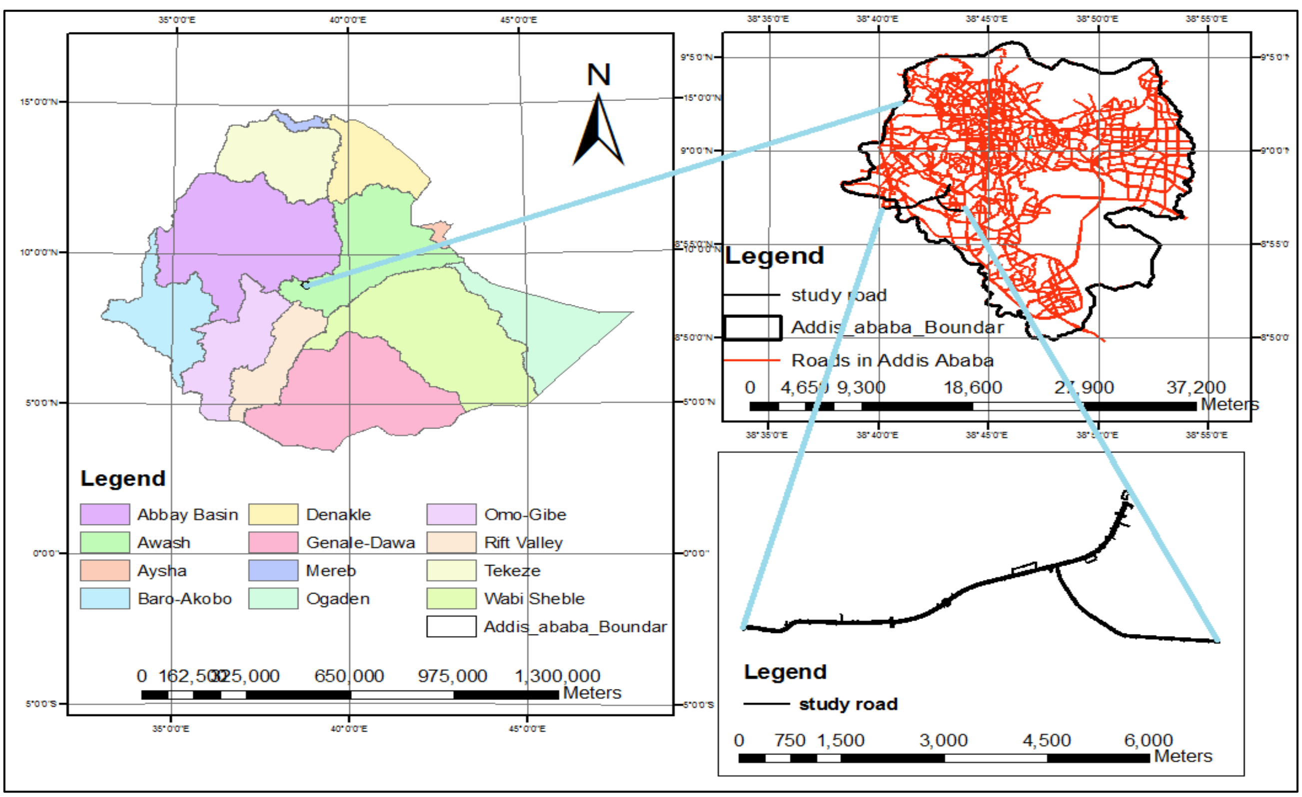


Figure 3-1.Location Map of Study Area **(MWIE, 2015)**

3.1.2. Climate

According to the National Meteorological Map of Ethiopia, the project area Addis Ababa is classified as “temperate” with an effective temperature of between 14 ^{0}c - 20 ^{0}c (mild), which is most of the time comfortable. The moisture index i.e., the ratio of precipitation to potential evapotranspiration is greater than 1.0 and can be described as humid.

The project areas receive most of its rainfall from June to September. The maximum rainfall occurs in the months of July and August.

The mean period of on-set of the “Kiremt Rains” is 20-30 May, while the mean period of cessation of the “Kiremt Rains” is 28 September-2 October. (AACRA, 2007)

3.1.2.1. RAIN FALL

Addis Ababa receives mean annual rainfall in the range of 918 - 1567mm, and the rainy period is between June and September; although occasional shower is expected in the months of March, April, May, October and November. The mean monthly rainfall get maximum during the months of July and August i.e. 250mm and 280mm and gets minimum during the months of November and December which is 8mm and 9mm respectively.

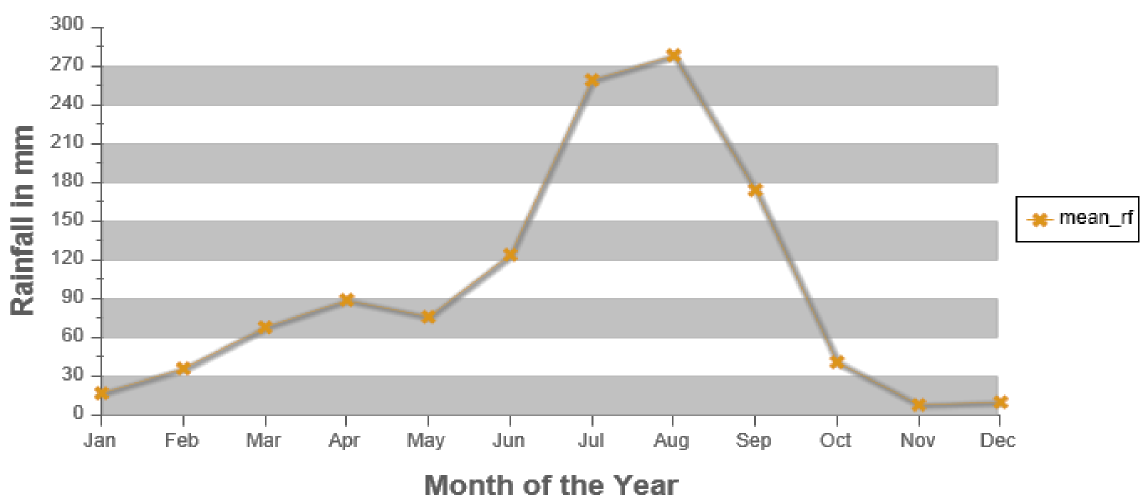


Figure 3-2. Rainfall chart of of Addis Ababa city (NMA,2015)

3.1.2.2. TEMPERATURE

For Addis Ababa area, the monthly temperature is maximum during the months of March and May, about 27.5^{0}C, and it is minimum in the months of November through January, 4.7^{0}C.

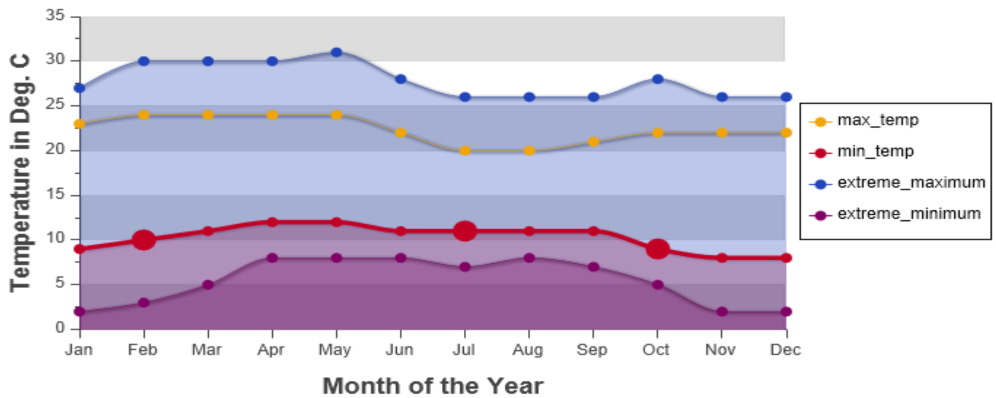


Figure 3-3. Temperature chart of Addis Ababa city (NMA).

3.2. Data Collection

To perform the study primary and secondary data were collected. The primary data was obtained by direct field investigation of operation, performance and maintenance of existing drainage structures and the secondary data was collected from different sources. The data's are:

- Design report of the road was obtained from Highway Engineers and Consultants
- Land use and land cover map of study area was obtained from Ethiopian Ministry of Water Irrigation and Energy.
- Soil map from Ethiopian Ministry of Water Irrigation and Energy.
- DEM of the area was obtained from Ethiopian Ministry of Water Irrigation and Energy.

The data were compiled, prepared and brought to analysis.

3.3. Hydrologic Study

The main purpose of hydrologic study is to compute the peak flood of road-crossing rivers or streams so as to determine the size of drainage structures along the road alignment.

The first step in doing any kind of hydrologic modeling involves extracting streams, delineating watersheds, and getting some basic watershed properties such as area, slope, flow length, stream network and density. Traditionally this was and still is being done manually by using topographic (contour maps). With the availability of digital elevation models (DEM) and GIS tools, watershed properties can be extracted by using automated procedures.

The processing of DEM to delineate watersheds is referred to as terrain pre-processing. There are several tools available online for terrain pre-processing. In this paper, Arc Hydro tools were used to process a DEM to delineate watershed characteristics, i.e. watershed size and shape, stream slope, stream length, land slope, stream network and some other watershed characteristics that collectively describe the drainage patterns of a basin. The results from this analysis can be used as input for the selected hydrologic method.

A DEM is a raster representation of a continuous surface, usually referring to the surface of the earth. The accuracy of this data is determined primarily by the resolution (distance between sample points).

3.3.1. Arc Hydro Catchment Modeling

The purpose of catchment modeling is to extract the watershed, watershed parameters and to prepare a dataset for further processing. DEM is required as an input for the catchment modeling. The catchment modeling was performed in Arc GIS environment with Arc Hydro 9 extension tool. Digital elevation model of Addis Ababa is obtained from Ethiopian Ministry of Water Irrigation and Energy, with 30m x 30m resolution. The DEM was Geo referenced in Global Mapper-V11 to the coordinate system of the study area, Adindan, UTM Projection Zone 37. The DEM for the specific area of study was clipped by using spatial analyst tool in Arc GIS 9.3 environment.

Arc Hydro is an Arc GIS-based system geared to support water resources applications. It consists of two key components: Arc Hydro Data Model and Arc Hydro Tools

In order to do this the Arc Hydro must be loaded in Arc Map. Steps to load the Arc Hydro tools in Arc Map are:

- Open a new empty map document in Arc Map.
- Right click on the menu bar to pop up the context menu showing available tools from the tools check the Arc Hydro Tools to add the toolbar to the map document.

3.3.1.1. Loading the Data to Arc Map

Open a new arc map and Click on the Add icon to add the raster data. In the dialog box, navigate to the location of the data; select the raster file containing the DEM and click on the Add button. Data need to be projected (not geographic) and with an extent sufficient to contain all the data that will be generated as part of the project The added file will then be listed in the Arc Map Table of contents and save the map document.

When opening a new Arc Map or Arc Catalog, the Arc Hydro configuration is set to the default configuration that usually points to the file *untitled.ahd* located in the Windows temporary location unless modified in Arc Catalog.

In ArcGIS9.3, the new configuration and target locations will always be generated when saving the new map, even if the Arc Hydro tools toolbox has not been explicitly added to the Arc Toolbox window. To ensure that a correct spatial reference is set for the output vector location, data setting the correct projected spatial reference should be added to the map before saving the new map document.

3.3.1.2. Terrain Preprocessing

Terrain preprocessing are functions dealing with Digital Elevation Model (DEM) processing. They are mostly used once in order to prepare spatial information for later use.

The purpose of terrain preprocessing is to perform an initial analysis of the terrain and to prepare the dataset for further processing. A Digital Elevation Model (DEM) of the study area is required as input for terrain preprocessing to identify the surface drainage pattern. Once preprocessed, the DEM and its derivatives can be used for efficient watershed delineation and stream network generation.

1. **Fill Sinks**

This function fills the sinks in a grid. If a cell is surrounded by higher elevation cells, the water is trapped in that cell and cannot flow. The Fill Sinks function modifies the elevation value to eliminate these problems.

2. **Flow Direction**

One of the keys to deriving hydrologic characteristics about a surface is the ability to determine the direction of flow from every cell in the grid. Flow direction takes a surface as input and outputs a grid showing the direction of flow out of each cell.

3. **Flow Accumulation**

This function computes the flow accumulation grid that contains the accumulated number of cells upstream of a cell, for each cell in the input grid.

Flow accumulation grid is extracted from the flow direction grid. The flow accumulation records the number of cells that drain into an individual cell in the grid. The flow accumulation grid is essentially the drainage area to a specific cell measured in grid units. The flow accumulation grid is the core grid used in stream delineation.

4. **Stream Definition**

This function computes a stream grid based on a flow accumulation grid and a user specified threshold. The cells in the input flow accumulation grid that have a value greater than the threshold are assigned a value of 1 in the stream grid. All other cells are assigned no data.

5. **Stream Segmentation**

This function creates a grid of stream segments that have a unique identification. A segment may either be a head segment, or it may be defined as a segment between two segment junctions. All the cells in a particular segment have the same grid code that is specific to that segment.

6. **Catchment Grid Delineation**

This function creates a grid in which each cell carries a value (grid code) indicating to which catchment the cell belongs. The value corresponds to the value carried by the stream segment or sink link that drains that area, defined in the input stream segment link grid (Stream Segmentation) or sink link grid (Sink Segmentation).

7. **Catchment Polygon Processing**

This function converts a catchment grid into a catchment polygon feature class.

8. **Drainage Line Processing**

This function converts the input Stream Link grid usually created with the Stream Segmentation function into a Drainage Line feature class. Each line in the feature class carries the identifier of the catchment in which it resides. Note that the function Flow Direction with Streams also generates the Drainage Line feature class based on the input Stream feature class.

9. **Adjoint Catchment Processing**

This function generates the aggregated upstream catchments from the "Catchment" feature class. For each catchment that is not a head catchment, a polygon representing the whole upstream area draining to its inlet point is constructed and stored in a feature class that has an "Adjoint Catchment" tag. This feature class is used to speed up the point delineation process.

The input Drainage Line and Catchment feature classes must contain the field GridID for a catchment and its associated drainage line shares the same GridID that is the ID of the corresponding link used to generate those features (from the stream link or link grid).

The three functions Catchment Polygon Processing, Drainage Line Processing and Adjoint Catchment Processing convert the raster data developed so far to vector format.

10. Drainage Point Processing

This function allows generating the drainage points associated to the catchments.

11. Longest Flow Path for Catchments

This function allows generating the longest flow paths associated to the catchments. This is required to speed up the generation of Longest Flow Paths.

12. Longest Flow Path for Adjoint Catchments

This function allows generating the longest flow paths associated to the adjoint catchments.

3.3.1.3. Watershed Processing

Arc Hydro toolbar also provides an extensive set of tools for delineating watersheds and sub watersheds. These tools rely on the datasets derived during terrain processing

The steps in Terrain Preprocessing need to be performed before the watershed delineation functions may be used. The preprocessing functions partition terrain into manageable units to allow fast delineation operations.

1. Batch Watershed Delineation

This function delineates the watershed upstream of each point in an input Batch Point feature class. The Arc Hydro tool Batch Point Generation may be used to interactively create the Batch Point feature class.

To create the Batch Point input file.

- Click on the  icon on the Arc Hydro Tools toolbar.
- Click with the mouse on the map to create a point at a location where you want to delineate a watershed.
- Fill in the fields Name and Description. Both are string fields

The Batch Done option indicates whether the Batch Watershed Delineation function will perform delineation for that point (0: delineate, 1: do not delineate).

The Snap On option indicates whether the Batch Watershed Delineation function will try to snap the point to the closest stream.

- **To perform a batch watershed delineation**

Select Watershed Processing → Batch Watershed Delineation.

Confirm that the input is Flow Direction Grid, Stream Grid, Catchment, Adjoint Catchment, and Batch Point. Then the output is the Watershed Point and Watershed.

In this study watersheds were extracted based on Batch point generation function, which is putting batch points on a position of streams crossing the road. Thus using the points as outlet point and delineating the area upstream of each point.

2. Watershed Characterization Functions

a. Drainage Area Centroid

This function generates the centroid of drainage areas as centers of gravity. However, if the center of gravity is not located within the polygon, the function will use as centroid the projection of the center of gravity on the polygon boundary (i.e. the nearest point on the boundary).

The function operates on a selected set of drainage areas in the input Drainage Area feature class. If no drainage area has been selected, the function operates on all the drainage areas.

b. Longest Flow Path

This function identifies and computes the length of the longest flow path in a selected set of drainage areas. If no drainage area has been selected, the function processes all the drainage areas.

c. Longest Flow Path for Watersheds

This function generates the longest flow paths for input watersheds more efficiently than the previous function because it relies on preprocessed data to speed up the process.

For this study watershed Characterization Functions helped to extract drainage area centroid, longest flow path and longest flow path for the watershed which were used as watershed parameters in doing hydrologic analysis.

d. Network Tools

If the dataset already has the geometric network with Hydro Edges and Hydro Junctions layers defined, one can directly use all the Attribute Tools.

e. Hydro Network Generation

This function allows converting drainage features into network features, and creating the associated geometric network. It also creates a relationship class (HydroJunctionHasCatchment) between the new Hydro Junction feature class (Hydro Junction) and the Catchment feature class that will be used subsequently.

f. Attribute Tools

If dataset already has the geometric network with Hydro Edge and Hydro Junction layers defined, you do not need to use the "Hydro Network Generation" tool. You can directly use the Attribute Tools.

g. Compute Local Parameters

This function allows computing area characteristics (e.g. average elevation, area, etc.) for selected polygon feature(s) in the input Drainage Area polygon feature class and storing them in the attributes table of the polygon layer. Examples of parameters that may be computed are:

- Area in square kilometer
- Average elevation in meter
- Maximum elevation in meter

- Minimum elevation in meter
- Relief (Difference between the maximum and the minimum elevations) in meter
- Average slope in percent

Select Attribute Tools ⟶ Compute Local Parameters.

Uncheck “Select all parameters” and then check Area M^2, ELEV Min, ELEV Max and stream Slope to select the parameters that will be extracted. Click OK.

Select “Watershed” as Drainage Area and click OK.

This function computes the specified parameters which are area in square meter, average elevation, elevation difference, slope 10% and slope 85% for the input watershed features and stores the results in the attributes table.

h. Up utilities

This function allows managing Arc Hydro project properties.

1. **Configure function parameters**: This function allows reconfiguring functions' parameters in the XML. Select the configuration the tools should use
2. **Additional utilities:** consists set of advanced configuration utilities
3. **Export to excel:** export the attributes of configured layers into an Excel spreadsheet

This function helped to export the Arc Hydro processed data in to Excel. After the parameters exported to Excel the next step was to estimate peak discharge of streams crossing the road using appropriate method recommended with ERA.

3.3.2. Hydrologic analysis

There are many methods available for hydrological analysis. According to ERA DDM, (2002), of these possible hydrologic methods only the Rational and SCS methods are applicable to the whole country Ethiopia.

Of this two methods selection criteria lies on the most important watershed property which is watershed area. It determines the potential runoff volume. The larger the watershed area, the greater the amount of surface runoff and, consequently, the greater the surface flows.

Watershed area plays an important role in setting methodology for analyzing surface runoff. The rational method is applicable for watershed having an area less than $0.8km^2$ while the SCS method is applicable for watershed having an area greater than $0.8km^2$. Determinations of which method to use for specific watershed was done after catchment area delineation on Arc Hydro tool which operate in Arc GIS environment.

3.3.2.1. Rational Method Peak Discharge Estimation

It is one of most commonly used equations for calculation of peak discharge for drainage area less than or equal to 80ha. The peak discharge for the rational method occurs when the entire watershed is contributing.

The rational formula estimate the peak rate of runoff at any location in a catchment as a function of the catchment area, runoff coefficient, and mean rainfall intensity for duration equal to the time of concentration (the time required for water to flow from the most remote point of the basin to the location being analyzed)

$$\boldsymbol{Q = 0.00278 * CIA}$$

Where:

Q = maximum rate of runoff, m^3/s

C = runoff coefficient representing a ratio of runoff to rainfall

I = average rainfall intensity for a duration equal to the time of concentration, for a selected return period, mm/hr.

A = catchment area tributary to the design location, ha

In this study the rational method is used for streams crossing the road having catchment area of $\leq 0.8Km^2$ based on ERA DDM.

3.3.2.1.1. Drainage area

Drainage area is the determining factor to choose which method is appropriate for peak discharge estimation. To estimate peak discharge in rational method the area should be ≤ $0.8Km^2$. For this study Watersheds having area ≤ $0.8Km^2$ are selected from GIS exported excel file. (*Ref. section 3.3.1)*

3.3.2.1.2. Return period

A design frequency (return period) was selected to match the structure’s cost, amount of traffic, potential flood hazard to property, expected level of service, political considerations, and budgetary constraints, and magnitude and risk associated with damages from larger flood events.

For this study a particular return period has been selected for the design based on AACRA DDM, 2004 which gives the design return period based on road class and major vs. minor drainage structure whereas, it doesn’t give the design return period for the specific structure. So that ERA DDM 2013 is also used to support the decision made to select design return period of specific structures.

As obtained from the design report of the road (High way Engineers and Consult, 2007) the study road is classified under urban principal arterial road. From Table 3-1 below for this type of road the return period for peak discharge estimation is 25 to 50 years return period.

Table 3-1. Return period for design of peak flood, AACRA DDM (HDS2, 1996)

Roadway Classification	Exeedance Probability	Return Period
Urban Principal Arterian Road	4% - 2%	25 – 50-year
Urban Minor Arterian Road	4%	25-year
Urban Collector Street System	10%	10-year
Urban Local Street System	20% - 10 %	5 – 10-year

In addition to this, the manual also states drainage system consists of major and minor systems. This study focuses on major drainage systems of which provide overland relief for storm water exceeding the capacity of minor systems. These stream crossings are designed to pass stream discharge of 25, 50 and 100 year return period.

3.3.2.1.3. Time of concentration

Time of concentration is the period required for water to travel from the most hydraulically remote point of the catchment area to the storm drain under consideration.

Methods for estimating time of concentration for this study were Time of Concentration for Defined Watercourses and Time of Concentration with Kirpich method. Both methods were compared and found that even if Kirpich method was criticized for making the time of concentration short resulting over estimated peak discharge but for this specific study Defined Watercourses time of concentration method is found to give shorter time of concentration for small area and the same result as kirpich method for relatively large area. So that Kirpitch time of concentration is used for this specific study. *(Ref section 2.3.1.3.1)*

3.3.2.1.4. Rainfall intensity

The rainfall intensity (I) is the average rainfall rate in mm/hr. for duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the catchment area, the rainfall intensity can be determined from Rainfall-Intensity-Duration curves.

For this study the rainfall intensity is determined from Rainfall-Intensity-Duration-Frequency curves of Addis Ababa Bole International Airport.

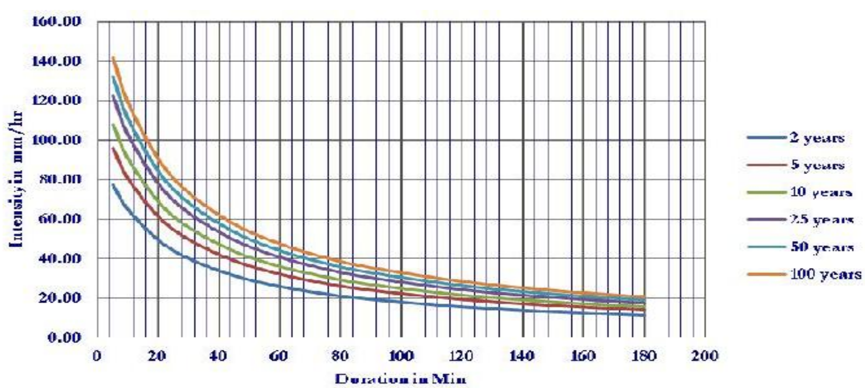


Figure 3-4. IDF curve for Addis Ababa Bole International Airport

3.3.2.1.5. Runoff coefficient

Runoff coefficient is the dimensionless value representing characteristics of the watershed that affects how much of rain become runoff. Coefficient selection is based on land use and soil conditions.

Runoff coefficient is theoretically restricted to the range 0.09 to 1. Considering future trends of the study area a runoff coefficient of 0.7 was used.

3.3.2.2. SCS Method

The method is applicable for catchment areas greater than 80 hectares (0.8km^2). The method can be used for drainage areas which include areas outside the highway pavement, as for roadside ditches and drainage systems which combine highway pavement drainage with other drainage.

Application of the method requires identification of hydrologic soil groups, watershed area, percent impervious, and overall slope.

3.3.2.2.1. Rainfall runoff equation

The SCS method uses the equations below to compute the depth of excess rainfall or direct runoff from the storm.

$$P_{en} = \frac{(P_n - I_a)^2}{P_n - I_a + S}$$

$$P_{en} = \frac{(P_n - 0.2S)^2}{P_n - 0.8S}$$

Where

P_{en} = Depth of direct runoff, mm

P_n = Depth of 24-hour precipitation for n year return period, mm

S = Retention, mm

$S = \frac{1000}{CN} - 10$ Where, S is in inches.

For this study depth of 24hr precipitation for n year return period obtained from ERA DDM 2013.

	24 hr Rainfall Depth (mm) vs Frequency (yr)							
Return Period Years	2	5	10	25	50	100	200	500
RR-A1	50.30	66.02	76.28	89.13	98.63	108.06	117.48	130.00
RR-A2	51.92	65.52	74.45	85.70	94.07	102.45	110.91	122.27
RR-A3	47.54	59.61	67.66	77.92	85.62	93.34	101.13	111.58
RR-A4	50.39	63.83	72.28	82.55	89.97	97.20	104.32	113.63
RR-B1	58.87	71.26	79.29	89.35	96.84	104.37	112.02	122.41
RR-B2	55.26	69.95	79.68	92.03	101.29	110.61	120.07	132.87
RR-C	56.52	71.04	80.54	92.52	101.48	110.50	119.66	132.06
RR-D	56.23	76.84	90.37	107.46	120.23	133.05	146.00	163.44

Note: RR- Rainfall Region

Table 3-2. 24Hr Rainfall depth vs. Frequency (ERA, 2013)

3.3.2.2.2. Curve Number

The SCS uses a combination of soil conditions, land-use (ground cover) and antecedent moisture condition to assign a runoff factor to an area.

The Hydrologic soil grouping for each catchment is identified from examination of available soil map from Ministry of Water, Irrigation and Energy Resources Metadata Bases (Shape files).

Ethiopian soil map shape file is added to Arc GIS and from arc tool box extract options is selected and by making the input feature Ethiopia soil shape file and external boundary as output feature the soli map for the study area is clipped.

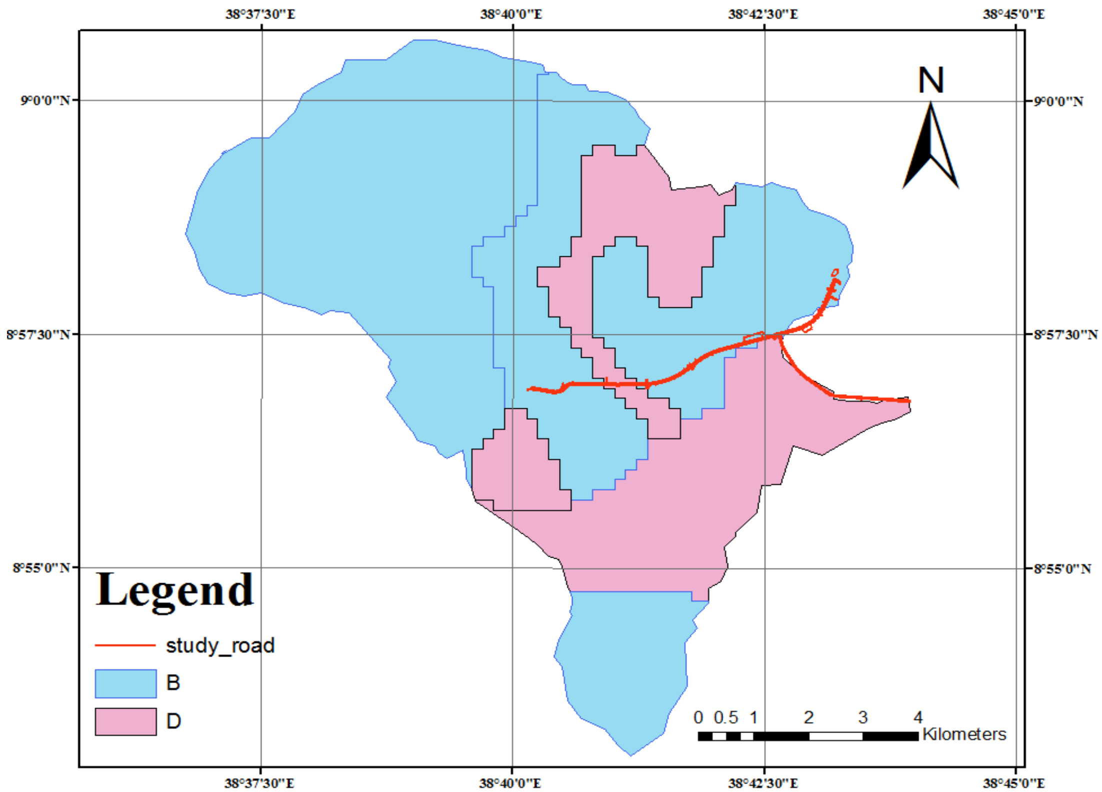


Figure 3-5. Soil map of the study area

Accordingly Eutric Nitisols and Chromic Luvisols found to be the dominant soil type throughout the area which is characterized as Hydrologic Soil Group B as per ERA DDM, 2002.

ERA drainage design manual also provides antecedent moisture conditions (AMC) in Ethiopia, dry for Region D1, wet for Region B1 and average AMC for all other regions. The manual also gives series of tables related to runoff factors according to average (normal) antecedent moisture condition (AMC II)

The study area Addis Ababa is located in Rain fall region A2 where in average AMC region so that the tables given in ERA DDM was used without any additional estimation.

During the field visit noticed that the area has similar land use, land cover and soil type. Using the field investigation data and considering future development the CN values was determined from the ERA DDM based on factors like cover type, land use and hydrologic soil groups, thus a representative CN value of 92 was selected.

3.3.2.2.3. Time of concentration

For this study Time of Concentration for Defined Watercourses method and kirpitch method were used to estimate the time of concentration.

$$T_c = \left(\frac{0.87L^2}{1000S_{av}}\right)^{0.385} \qquad (3\text{-}1)$$

Where:

T_c = time of concentration (hours).

L = hydraulic length of catchments measured along flow path from the catchment boundary to the point where the flood needs to be determined (km).

S_{av}= average slope (m/m).

The formula for determining the slope according to the 1085-slope methods reads:

$$S_{av} = \frac{H_{0.85L} - H_{0.10L}}{1000 * 0.75L} \text{ or } S_{av} = \frac{H}{1000 * 0.75L} \qquad (3\text{-}2)$$

Where:

S_{av} =average slope (m/m)

$H_{0.10L}$ =elevation height at 10% of the length of the watercourse (m)

$H_{0.805L}$ = elevation height at 85% of the length of the watercourse (m)

L = length of watercourse (km)

H = $H_{0.805L}$ - $H_{0.10L}$ (m)

3.3.2.2.4. SCS Unit Hydrograph Peak Discharge Estimation Method

The peak discharge of streams crossing the road was estimated by SCS unit hydrograph method rather than SCS graphical peak discharge estimation in order to avoid error in reading graphs.

The lag La of the peak flow, time from the centroid of rainfall excess to the peak of the hydrograph, is assumed to be 0.6tc. Then the time of rise Tp to the peak of the hydrograph is

$$T_p = 0.5D + 0.6T_c \qquad (3\text{-}3)$$

$$\boldsymbol{Q_p} = \frac{\mathbf{0.208} * \boldsymbol{A} * \boldsymbol{p_e}}{\mathbf{0.5}\boldsymbol{T_c}^{\mathbf{05}} + \mathbf{0.6}\boldsymbol{T_c}} \qquad (3\text{-}4)$$

Where:

Q_p = peak discharge (m^3/s)

P_e = the excess rainfall depth (mm)

A = watershed area (km^2)

T_c= time of concentration (hr)

D = duration of excess rainfall (hr)

Peak discharge of streams crossing the road is estimated using both rational and SCS method. The results were compared with the hydrologic result of the original design document. The next step was to determine the opening requirement of this discharge.

3.4. Hydraulic Study

For this study the opening requirement of structures to pass the coming flow is determined by using Manning's formula. This method is used for the design flood levels at crossing sites after the design discharges have been estimated by one of appropriate hydrological methods.

$$\boldsymbol{Q} = \boldsymbol{K}\frac{\mathbf{1}}{\boldsymbol{n}} * \boldsymbol{A} * \boldsymbol{R}^{\frac{2}{3}} * \boldsymbol{S}^{\frac{1}{2}} \qquad (3\text{-}5)$$

Where:- Q = Discharge in [m^3/sec]

R = Hydraulic mean depth [m] = A/P

A = Cross-sectional flow area [m^2]

P = Wetted perimeter [m]

S = Longitudinal bed slope [%]

n = Manning's roughness coefficient

K = constant 1 for SI and 1.49 for English unit.

This equation is applicable for uniform open channel flow in which the water flows in a constant rate, with a constant slope, size, shape and roughness. For uniform flow, equilibrium exists along a partially full pipe or channel which is the common flow condition in urban drainage. Under this situation the water will flow with a constant depth.

The energy consumed by friction between the liquid and the pipe or channel wall is in balance with the fall along the pipe length. This equilibrium depth is referred to as ***normal depth***.

In urban drainage partially full flow is the common flow condition. For this flow to occur, the water shall flow with a constant depth. So that this equilibrium depth needs to be determined in order to maintain partially full flow.

The procedure to determine this normal depth for pipe flow and channel flow is the same. For flow in a channel with specified Q, n, and S, the right hand side of the equation has a constant value that can be calculated. Whereas the left hand side of the equation is written as a function of the depth of flow, y, for a specified channels shape and size.

An iterative solution is performed to find the value of **y** that makes the two sides of the equation equal.

$$A\left(R_h^{\frac{2}{3}}\right)/n = Q/1.49\left(S^{1/2}\right) \qquad (3\text{-}6)$$

For this study equations for calculating the cross sectional area of flow, A, the wetted perimeter, P and hydraulics radius R_h for partially full pipe flow is used.

$$r = D/2 \qquad (3\text{-}7)$$

$$\theta = 2arccos\left(\frac{r-y}{r}\right) \qquad (3\text{-}8)$$

$$A = \pi r^2 - r^2\left(\frac{\theta - sin\theta}{2}\right) \qquad (3\text{-}9)$$

$$P = 2\pi r - r\theta \qquad (3\text{-}10)$$

$$R_h = \frac{A}{P} \qquad (3\text{-}11)$$

Keeping design discharge and design diameter constant and using the above equations iteration is made to get the constant depth that the flow will attain to keep uniform flow. The procedure is the same with box culvert.

3.4.1. Design consideration

The design considerations for the pipe and box culvert are listed below. They are as per AACRA and ERA DDM.

3.4.1.1. Culvert Design slope

Culvert grades should match that of the natural channel slope. The minimum grade for the culvert should be 0.5%. Flatter grades may be prone to siltation. The maximum grade should be chosen to limit the pipe full flow velocity to a value less than or equal to 6 m/s to avoid scour (AACRA DDM, 2004).

3.4.1.2. Culvert minimum Diameter

Apart from pavement drainage whose inlets are small curbs and grates, culverts are prone to blockage with large debris. To overcome the risk the minimum pipe diameter as per AACRA DDM 2004, for culvert crossings should be 600mm.

3.4.1.3. Free board

An important factor on designing culvert is free board. It's the vertical distance from the water surface to the structure. The free board according to ERA DDM, 2002 is shown below.

Table 3-3. Design discharge vs. free board, ERA DDM, (2002)

Discharge, Q (m^3/s)	**Freeboard (m)**
0 – 3.0	0.3
3.0 -30	0.6
30 -300	0.9
> 300.0	1.2

By considering the above situations a trial pipe size and culvert cross section were used and iterated to get the normal depth of flow. After that a recommended free board is considered to determine the required size of structure.

4. RESULTS AND DISCUSSION

4.1. Hydrologic study result

4.1.1. Arc hydro Catchment modeling result

The catchments for streams crossing the road were determined by processing 30mx30m resolution satellite DEM in ARC GIS 9.3 environment with Arc hydro tool 9. This gave watersheds with different watershed characteristics, i.e. watershed size, shape, stream slope, stream length and land slope. The study road is classified in to two sections. Road section 1 is *Mikael taxi station-Jemo mestawet fabrica-Sebeta* and Road section 2 is *Lebu roundabout to Jemo mestawet fabrica*.

1. **Results of Catchment area delineation for Road section 1**

The Arc Hydro analysis for road section 1 *(Mikael taxi station-jemo mestawet fabrica-sebeta)* gave 12 catchments with different watershed characteristics. From watershed delineation and parameter extraction it was found that the catchment area of streams crossing the road are streams having area less than $0.8Km^2$ and the others having area greater than $0.8Km^2$. So that using the idea of AACRA drainage design manual, a method was selected for estimating peak discharge of streams crossing the road in Rational and SCS method.

From the analysis stream parameters such as stream length, stream slope, stream 10-85 elevation (*i.e.* Elevation height at 10% of the length of the watercourse and elevation height at 85% of the length of the watercourse) and stream slope in 10-85 method is obtained. These parameters were further used in empirical peak discharge estimation methods.

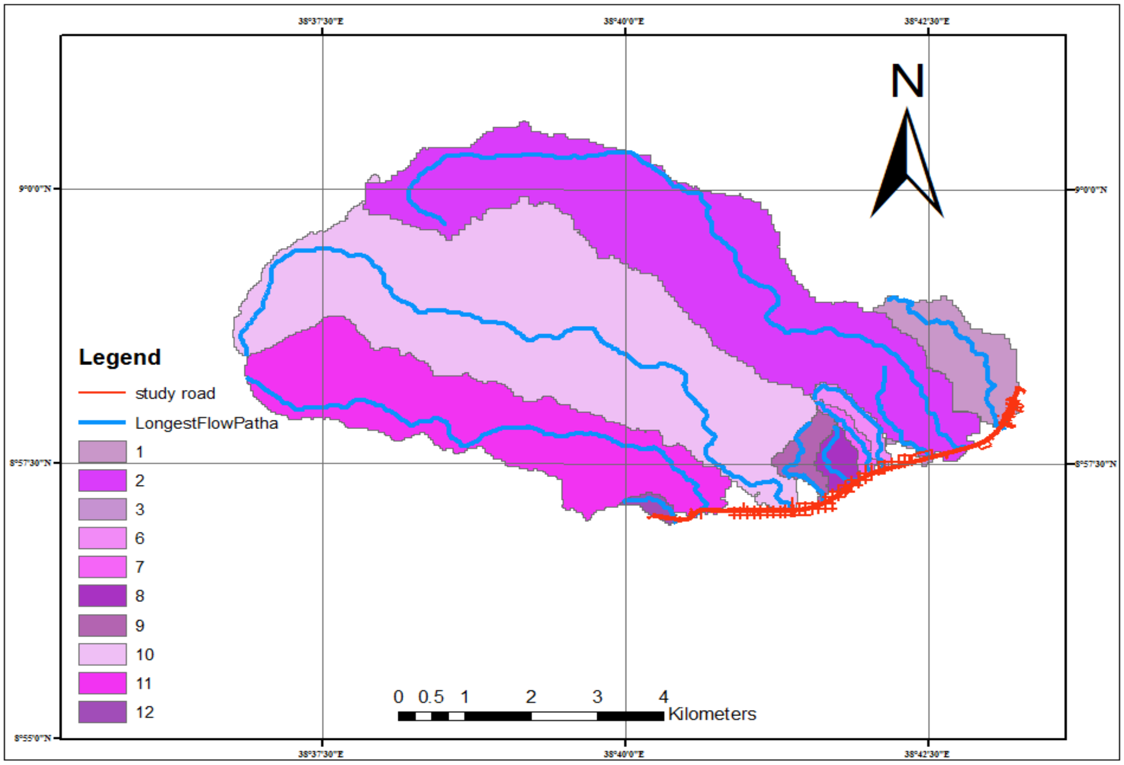


Figure 4-1. Watersheds for Road section 1

Figure 4-1, shows catchments of streams together with stream line crossing road section 1.

Table 4-1. Watershed parameters for Road section 1

Drainage Area				
Cach ID	MinElev	MaxElev	area_(km2)	method_Slectn
1	1543	1952	2.372	SCS
2	1909	2565	16.418	SCS
3	2316	2688	0.184	Rational
4	2728	2773	0.2182	Rational
5	1084	2732	0.1431	Rational
6	2439	2855	0.688	Rational
7	2605	2796	0.3114	Rational
8	2772	2952	0.49224	Rational
9	2697	3008	0.66378	Rational
10	1441	3135	21.0116	SCS
11	2108	3157	9.3255	SCS
12	3066	3215	0.2154	Rational

Table 4-2. Stream parameter for Road section 1

Stream parameter					
Cac' id	Length_km	Elev10	Elev85	Slp	Slp1085
1	3.31503	1231.838	1287	0.01664	0.022
2	12.8691	2253	2632.61	0.0295	0.039
3	2.396	2232.083	2275.57	0.018	0.024
4	0.774	2236	2245	0.012	0.016
5	1.344	2250.4	2621.635	0.028	0.037
6	2.289	2247.49	2418	0.075	0.099
7	1.681	2243.06	2401.988	0.095	0.126
8	1.416	2237.72	2338.704	0.071	0.095
9	2.153	2251	2302.141	0.024	0.032
10	13.274	2264.47	2779.066	0.039	0.052
11	9.089	2282.31	2750.652	0.0516	0.069
12	1.113	2272.65	2340.642	0.061	0.082

2. Results of Catchment area delineation for Road section 2

The arc hydro analysis for road section 2 *(Lebu roundabout to jemo mestawet fabrica)* gave catchments with different watershed characteristics. Figure 4.2 shows the catchments.

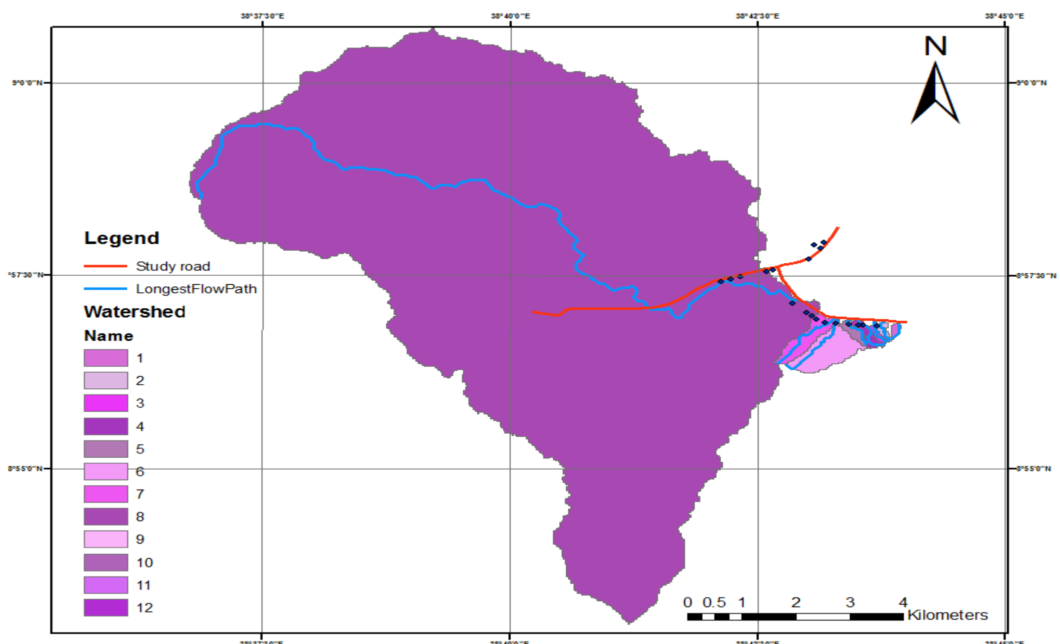


Figure 4-2. Watersheds for Road section 2

Table 4-3. Watershed parameters for Road section 2

Drainage area				
OBJECTID	area_km2	MinElev	MaxElev	method_Slectn
1	77.418	1408	7017	SCS
2	0.17822	3885	4236	Rational
3	0.15584	3912	4137	Rational
4	0.1165	3977	3977	Rational
5	0.7822	4074	4268	Rational
7	0.3142	4138	4380	Rational
8	0.3123	4139	4248	Rational
9	0.3614	4040	4040	Rational
10	0.0373	4140	4140	Rational
11	0.0224	4062	4062	Rational
12	0.27	4063	4063	Rational

Table 4-4. Stream parameters for road section 2

Stream parameter					
Cach ID	Length_km	Elev10	Elev85	Slp	Slp1085
1	16.5	2236.695	2734.2	0.030143	0.040
2	0.255	2232.1	2340.5	0.042462	0.057
3	0.166	2233.9	2378.8	0.087217	0.116
4	0.745	2236	2265	0.038927	0.052
5	1.437	2243.3	2416.2	0.120304	0.160
7	1.527	2239	2290.6	0.033344	0.045
8	1.546	2255	2307.8	0.075181	0.100
9	0.703	2252	2314.9	0.126951	0.169
10	0.495	2248.6	2320.4	0.166089	0.222
11	0.432	2248.6	2320.4	0.166089	0.222
12	0.432	2240.7	2311.1	0.158212	0.211

Table 4-3 and 4-4 shows watershed and stream parameters for catchments of streams crossing the road.

After getting the catchment parameters for the watersheds the next step was to estimate the peak discharge in which each catchment is producing using empirical methods. Rational peak discharge estimation method is used for catchment area $\leq 0.8Km^2$ and SCS method is used for catchment area $\geq 0.8Km^2$.

4.1.2. Peak discharge of streams crossing the road

For the return period determined in *section* 3.3.2.1.2 the peak design discharge were computed using Rational and SCS peak discharge estimation methods for each road section. The detailed calculation and result for hydrologic analysis is shown below.

4.1.2.1. Rational Peak Discharge

The rational formula estimate the peak rate of runoff: at any location in a catchment as a function of the catchment area, runoff coefficient and mean rainfall intensity for duration equal to the time of concentration. According to the methodology the peak discharge for each road section is estimated.

➢ **Rational Peak discharge for road section 1**

Catchment area A=0.2182km^2, as determined from catchment modeling

Runoff coefficient C, for catchment 4 = 0.7 as determined in section *3.3.2.1.5*
Time of concentration Tc =18min, as determined in section3.3.2.1.3

Intensity of rain fall is determined from IDF curve of Addis Ababa Bole international airport for 25 year return period as determined in section *3.3.2.1.2* for 18min of time of concentration I_{25}= 81mm/hr
$Q = 0.278 * CIA$, Q = 0.278* 0.7*81*0.2182= 3.44m^3/s

The same procedure is followed for all the catchments in both road sections having area of catchment ≤ 0.8km^2. The result of the calculation is shown in Table **4-5** and *Table 4-7* below for road section 1 and road section 2 respectively. Whereas the detailed design spread sheet is shown in *Appendix 2.*

4.1.2.2. SCS Peak Discharge Estimation

This method is applicable for catchment areas greater than 80 hectares (0.8km^2). It's developed by the U. S. Soil Conservation Service for calculating rates of runoff. It requires the same basic data as the Rational Method: catchment area, a runoff factor, time of concentration, and rainfall.

- **SCS peak discharge for road section 1**

Using the procedure for calculating peak discharge in SCS method for catchment 1:

Catchment area determined from catchment modeling = 2.372Km^2

Curve number = 92 as discussed in section *3.3.2.2.2*

Potential retention, $S\ = 254\left(\frac{100}{CN} - 10\right)$= 22mm

Rainfall abstraction, I_a= 0.2*S = 4.417mm

Depth of 24 hour precipitation for 25 year return period = 85.7mm from *Table 3-2*

From SCS rainfall runoff equation $P_{en} = \frac{(P_n - I_a)^2}{P_n - I_a + S}$, depth of direct runoff = 97.12mm

Time of concentration is estimated using kirpich method, $tc\ = \frac{KL^{0.77}}{S^{0.385}}$= 0.724hr

SCS peak discharge is estimated by $Q_p = \frac{0.208 * A * p_e}{0.5{T_c}^{05} + 0.6T_c}$ = 55.76m^3/s

Using the same procedure peak discharge for the rest of catchments having area ≥ 0.8Km^2 were performed. The result of the analysis is presented in

Table **4-6** and *Table 4-8* below for road section 1 and road section 2 respectively. The detailed design spread sheet is shown in *Appendix 3.*

Table 4-5.Rational peak for road section 1

Catchment parameter			**Stream parameter**				**Peak discharge estimation**
Cach ID	area(km^2)	C	Length km	Slp(m/m)	Tc(min)	I_{25}	Q_{design} (m^3/s)
3	0.1837	0.7	2.396	0.0242	22.14	67	2.39
4	0.2182	0.7	0.774	0.0116	18.173	81	3.44
5	0.1431	0.7	1.344	0.0368	17.811	98	2.73
6	0.688	0.7	2.29	0.0745	20.483	78	10.44
7	0.3114	0.7	1.681	0.0945	14.729	90	5.45
8	0.49224	0.7	1.416	0.0713	14.382	90	8.62
9	0.66378	0.7	2.153	0.0238	30.326	60	7.75
12	0.21536	0.7	1.113	0.061	12.677	98	4.11

Table 4-6. SCS peak discharge estimation for road section 1

Catchment parameter			Stream parameter		Peak discharge estimation			
Cach ID	Area (km^2)	CN	Length (km)	Slope (m/m)	P25	Pe25	Tc (hr)	Q $_{design}$ (m^3/s)
1	2.372	92	3.31503	0.022187	85.7	97.12	0.724	55.76
2	16.418	92	12.8691	0.03933	85.7	97.12	1.65	203.33
10	21.0112	92	13.274	0.0516895	85.7	97.12	1.5197	277.73
11	9.3256	92	9.0892	0.0687032	85.7	97.12	1.0175	168.97

Peak discharge for road section 2

Peak discharge of streams crossing road 2 are estimated in rational and SCS peak discharge estimation methods based on drainage area. The result of the analysis is shown in *Table 4-7* and for rational peak and *Table 4-8* SCS peak discharge estimation result respectively. While the detailed hydrologic analysis results of road section 2 is shown in *Appendix 4* and Appendix 5.

Table 4-7. Rational peak for road section 2

Catchment parameter				Stream parameter		Peak discharge estimation	
Catc.t ID	Area (km^2)	C	I_{desig}	Length (km)	Slp (m/m)	T_c (min)	Q $_{design}$ (m^3/s)
2	0.178	0.7	58	0.255	0.057	24.75	1.734
3	0.156	0.7	77	0.166	0.116	15	1.82
4	0.1167	0.7	77	0.745	0.052	15	1.75
5	0.7822	0.7	77	1.437	0.16	15	11.72
7	0.3142	0.7	77	1.546	0.159	15	4.16
8	0.3123	0.7	68	0.703	0.044	18.47	4.68
9	0.3614	0.7	77	0.495	0.1	15	5.41
10	0.0373	0.7	77	0.432	0.169	15	0.56
11	0.0224	0.7	77	0.432	0.221	15	0.34
12	0.2737	0.7	77	0.445	0.221	15	4.10

Table 4-8. SCS peak discharge for road section 2

Catchment parameter			Stream parameter		Peak discharge estimation	
Cach.t ID	Area (km^2)	CN	Legt (km)	Slp (m/m)	Tc (hr)	Q_{design} (m^3/s)
1	77.42	92	16.51	0.0402	3.383	482.3

4.2. Hydraulic study result

For this study the hydraulic analysis to determine the opening requirement of structure to pass the coming flow is determined by Manning's formula. By doing the constants of the equation (i.e. Q, n and S) in one side and writing the other (i.e. A and R) as a function of flow depth tried to iterate until a constant depth is attained which can give both side of the equation equal. After the constant depth of flow is attained the opening requirement of structure is determined by adding the free board requirement for box culverts based on design peak discharge. *Table* **4-9** and *Table 4-10* show hydraulic analysis result of pipe culvert for road section 1 and 2 respectively. The detailed analysis spread sheet is shown in *Appendix 7*

Table 4-9**.** Pipe culvert at road section 1

Cach't Id	Q_{design} (m^3/s)	design slope (m/m)	Diameter (m)	No of pipe
3	2.4	0.030	1.2	1
4	3.44	0.005	1.2	2
5	2.73	0.005	1.2	1
7	5.45	0.005	1.2	2
12	4.11	0.005	0.9	3

Table 4-10. Pipe culvert at road section 2

Cach't Id	Q_{design} (m^3/s)	Slope (m/m)	D(m)	No of pipe
2	1.73	0.005	1.2	1
3	1.82	0.005	0.9	2
4	1.75	0.005	1.2	1
7	4.16	0.005	1.2	3
8	4.68	0.005	1.2	3
9	5.42	0.005	1.2	3
10	0.56	0.005	0.9	1
11	0.34	0.005	0.9	1
12	4.10	0.005	1.2	2

Table 4-11 and *Table 4-12* show the hydraulic analysis result of box culvert at road section 1 and 2 respectively. The detailed design spread sheet is shown in *Appendix 6.*

Table 4-11. Box culvert for road section 1

Cacht id	Q design (m^3/s)	Slope (m/m)	Yo (m)	free board (m)	design depth of structure	span	No. Barrel
1	55.76	0.022187	2.06932	0.9	3	2	2
2	203.33	0.03933	4.92855	0.9	3	4	2
6	10.44	0.0745	0.712362	0.6	1.3	2	1
8	8.62	0.0713	0.6212043	0.6	1.2	2	1
9	7.75	0.0238	0.7514095	0.6	1.5	2	1
10	277.73	0.0517	2.4693583	1.2	3.7	4	2
11	168.97	0.0687	2.8263568	0.9	3.7	2.5	2

Table 4-12.Box culvert for road section 2

Cach id	Q design (m^3/s)	Slope (m/m)	Yo (m)	free board (m)	design depth structure	span	No. of barrel
1	482.3	0.0402	2.6	1.2	3.8	4.5	3
5	11.7	0.1604	0.5135	0.6	1	2	1

4.3. Comparison between the Original Design and Redesigned

While comparing catchment modeling, peak discharge estimation and hydraulic capacity determination for this study with the original design report of the road, variations were obtained.

- **Drainage area variation:** Comparing catchment modeling result obtained for road section 1 and 2 with the original design showed that catchment area of streams which cross the road is underestimated. *Table 4-13* shows the comparison and amount undermined for road section 1 from Mikael taxi station to Jemo Mestawet Fabrica since the original hydrologic and hydraulic report for the rest of the road section was not available.

Table 4-13. Drainage area and stream length comparison

Catch ment no	Area result of this study (km^2)	Area result of HEC (km^2)	Area underes timated (%)	Stream length this study (km)	Stream length result for HEC (km)
Drainage area and stream length comparison for road section 1					
1	2.372	2.213	6.7	3.32	2.7
2	16.42	15.65	4.69	12.87	11.0
3	0.184	0.1	45.65	2.396	0.45
4	0.2182	Not considered	-	0.774	-
5	0.1431	Not considered	-	1.344	-
Drainage area and stream length comparison for road section 2					
1	77.42	61.15	21.02	16.51	16.5
2	0.178	Not considered	-	0.255	-
3	0.156	Not considered	-	0.166	-
4	0.1167	0.036	69.2	0.745	0.182
5	0.7822	0.746	4.63	1.437	0.964
7	0.3142	0.1	68.2	1.546	0.45
8	0.3123	0.25	20	0.703	1.25
9	0.3614	0.32	11.5	0.495	0.625
10	0.0373	Not considered	-	0.432	Not considered
11	0.0224	Not considered	-	0.432	Not considered
12	0.2737	0.22	7.2	0.445	0.57

As one can see from the Table 4-13 area and length of stream are underestimated. This is due to drainage area delineation on the original design of the road was performed manually from topographic map available which is labor intensive, inefficient activity which resulted underestimation of drainage area of streams crossing the road and length of stream. In addition to this two catchment i.e. catchment 4 and 5 in road section 1, four catchments in road section 2 i.e. catchment 2, 3, 10 and 11 were not considered. The hydrologic analysis report of the road section by High way Engineers and Consultant (HEC) is shown in *Appendix 8* and *Appendix 9* for road section 1 and 2 respectively.

- **Time of concentration variation:** this may be due to length of streams from remotest point to crossing point was measured manually from paper map for the original design which resulted wrong estimation of stream length and there by varying time of concentration.

Table 4-14. Time of concentration result comparison of this study with the HEC result

Time of concentration variation for road section 1			
Catchment no	Time of concentration as per HEC (min)	Time of concentration for this study (min)	Amount underestimated (%)
1	37	43.4	14.75
2	15	99	85
3	19	22.1	14
4	-	18.2	-
5	-	17.8	-
Time of concentration variation for road section 2			
1	170	203	16.3
2	-	24.748	-
3	-	13.47	-
4	8	15	46.7
5	15	17	11.8
7	11	18.47	40.5
8	15	26	42.3
9	9	15	40
10	-	15	-
11	-	15	-
12	8	15	46.7

- **Intensity of Rainfall variation:** it’s known that for rational method rainfall intensity and time of concentration are related i.e. variation in time of concentration will result variation in intensity.
- **Variation in curve number:** the CN in the original design of the road is used to be 88, whereas this value is insufficient while considering future land use change. So that a CN of 92 was used for this study.
- **SCS peak flow Variation:** variation in drainage area and variation in curve number used, as mentioned above resulted variation in peak discharge of streams crossing the road.
- **Rational peak flow variation:** rational peak flow is a function of drainage area, intensity and runoff coefficient so that the rational peak from the original design of the road drainage is found to be underestimated since the drainage area is underestimated.

Table 4-15 shows comparison between peak discharge of streams crossing the road obtained by HEC and this study. As shown in the table the discharge is underestimated. This is due to drainage area variation, land use land cover change and curve number used.

Table 4-15. Peak discharge comparison

Peak discharge variation for road section 1			
Catchment no	Design discharge for this study (m^3/s)	Design discharge by HEC, (m^3/s)	Q underestimated (%)
1	55.76	37	33.6
2	203.33	56	72.5
3	2.4	0.97	59.60
4	3.44	Not considered	-
5	2.73	Not considered	-
Peak discharge variation for road section 2			
1	482.3	451	6.43
2	1.73	Not considered	-
3	1.82	Not considered	-
4	1.75	0.54	69
5	11.72	8.3	29.2
7	4.16	1.11	73
8	4.68	2.09	55
9	5.41	3.6	33.5
10	0.56	Not considered	-
11	0.34	Not considered	-
12	4.10	2.75	33

- **Hydraulic variation:**

  It's obvious that peak discharge variation will result opening requirement variation. The same is true here since peak discharge is underestimated as discussed above opening requirement is also under estimated which resulted flooding. In addition, at stations where there is relatively equal estimation of flow and opening requirement the flood still exists it is because the roughness of the structure is changed due to large wastes are dumped and blocked the structures which minimize the flow area and result flood.

Table 4-16 shows opening requirement estimated with HEC and this study. Additionally the table shows the correct structures which must be provided in every stream crossing other than the one provided with the original design since the existing structures are not handling properly the coming flood. The structure designed at catchment 5 is a box culvert in the original design but the constructed structure is a pipe culvert with 1.2m diameter.

Table 4-16. Opening requirement comparison

Opening size requirement for road section 1 of this study				
Catchment no	Design depth of structure	Span/diameter (m)	No. Barrel	Type of structure
1	3	2	2	Box culvert
2	3	4	2	Box culvert
3		1.2	1	Pipe culvert
4		1.2	2	Pipe culvert
5		1.2	1	Pipe culvert
Opening size requirement for road section 1 as per HEC				
1	2.5	4.5	1	Box culvert
2	2.25	4	2	Box culvert
3	-	1	1	Pipe culvert
4	Not considered			
5	Not considered			
Opening size requirement for road section 2 of this study				
1	3.8	4.5	3	Box culvert
2	1	2	1	Box culvert
3	1	2	1	Box culvert
4	-	1.2	1	Pipe culvert
5	1	2	1	Box culvert
7		1.2	3	Pipe culvert
8		1.2	3	Pipe culvert
9		1.2	3	Pipe culvert
10		0.9	1	Pipe culvert
11		0.9	1	Pipe culvert
12		1.2	2	Pipe culvert
Opening size requirement for road section 2 as per HEC				
1	3.25	4.5	3	Box culvert
2	Not considered	-	-	
3	Not considered	-	-	
4		1	1	Pipe culvert
5	1.75	2	1	Box culvert
7		1	1	Pipe culvert
8		1	1	Pipe culvert
9		1.2	1	Pipe culvert
10	Not considered	-	-	-
11	Not considered	-	-	-
12		1.2	1	Pipe culvert

4.4. Design versus construction

Considering evaluation of design versus construction found that the construction for road section 1 at two stations is not as per the design. The original design proposed double box culvert 2(4.5m*2.5m) at station 1+890 and at station 2+903, during the field survey noticed that there is double pipe culvert at this stations which can't handle the coming design flow thus result flooding.

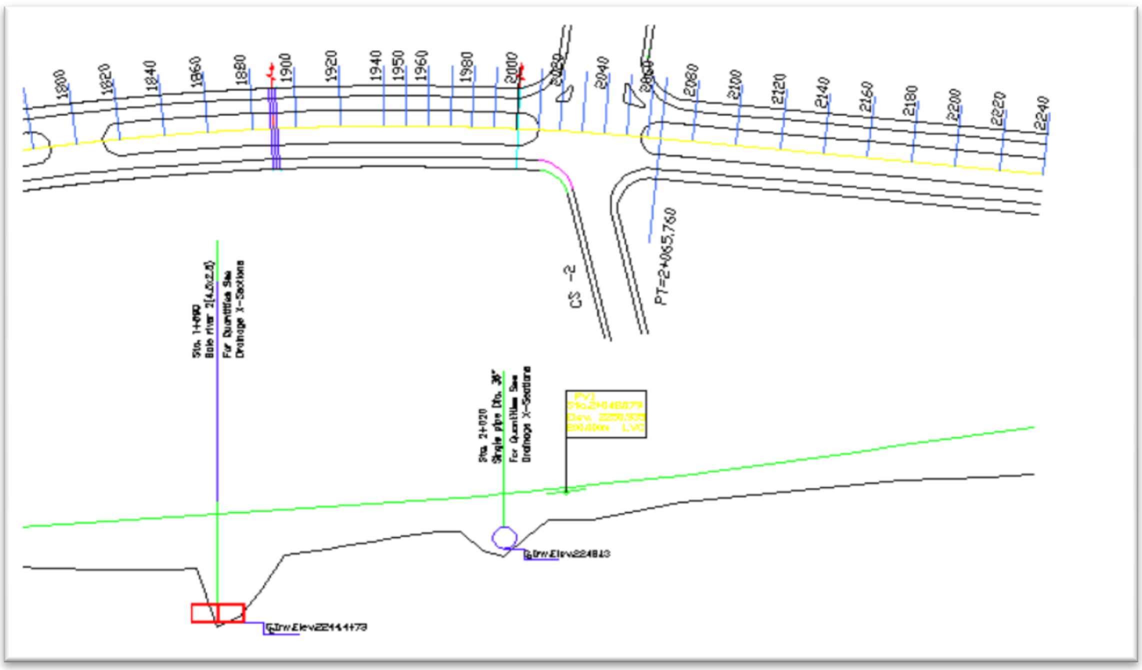


Figure 4-3.Proposed double box culvert at station 1+890

At this station the structure is not handling the flow so that the structure is reconstructed but still could not handle the flow so that another storage structure is constructed to stabilize the flow. The photo below shows the reconstructed structure and upstream storage structure.

Photo 4-1. Reconstructed structure and storage structure to stabilize the flow.

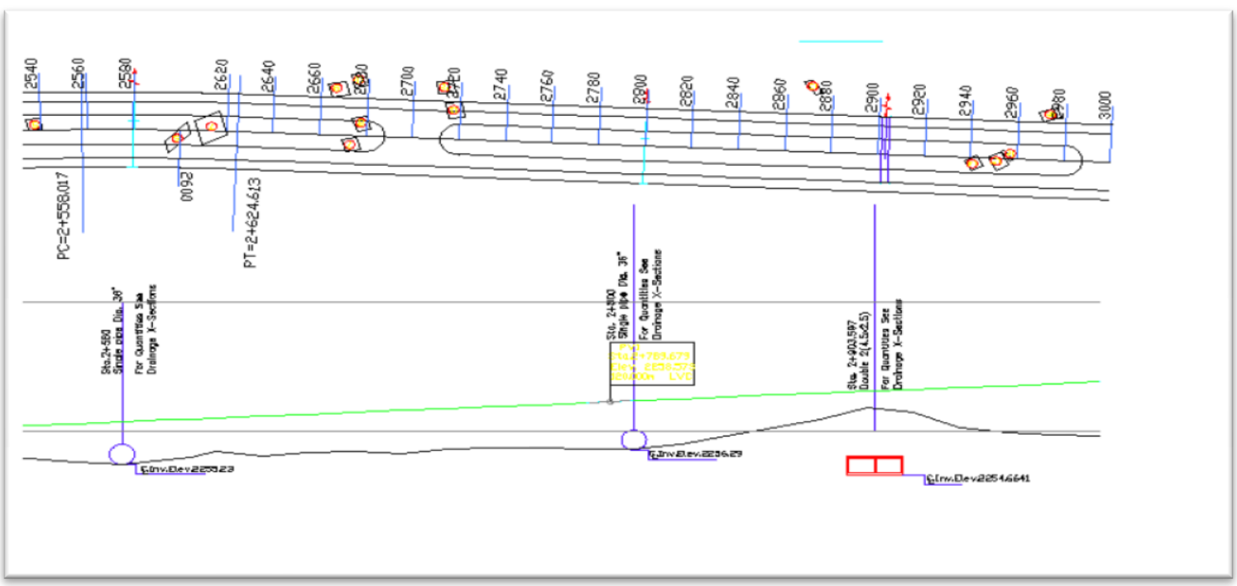


Figure 4-4. Proposed double box culvert at station at 2+903

4.5. Maintenance of Road surface and Drainage structures

As observed during field investigation there is no regular maintenance of road and drainage structures. Most of stream crossings are full of garbage and sediment at many places which increase roughness thereby reducing effective carrying capacity of structure and prevent normal flow of water in the culvert resulting flood.

Photo 4-2. Existing situation of Pavement and Drainage Structure on the Study Road.

4.6. Validating Culvert Size Based On Culvert Design Method

The recommended culvert sizes obtained from this study were checked in terms of Culvert design method. The design method was based on the location of the control (inlet or outlet). The design procedure was to use the recommended culvert size, material and calculate the headwater elevation for both inlet and outlet control. The higher of the two is designated as the controlling headwater elevation. The controlling headwater elevation is compared to the desired design headwater which is usually governed by overtopping considerations to determine the assumed culvert size is acceptable. So that the procedure is repeated for all culverts and the result is shown in Table 4-17 below.

Table 4-17. Recommended culvert size against culvert design Method

Head Water Calculation																			
Design opening Size								Inlet Control		Outlet Control							Type of control	Discharge Over the road way	
design clear depth	span	no.barrel	ow per barr	A	slab tickness	fill heghtt	Hwov	Hw/D	Hwi	Tw	dc(<D)	(dc+D)/2	ho	ke	H	Hwoi		HWr	Qr
3	2	2	13.94	6	0.25	0.3	4.25	1.01	3.0	2.6	2.7	2.9	2.9	0.2	0.40	2.99	inlet control	-1.2	0
3	4	2	25.42	12.4	0.32	0.3	4.57	1.02	3.2	3.1	4.0	3.6	3.6	0.2	1.47	4.57	outlet control	0.0	0
2	2	1	5.22	4	0.25	0.3	4.25	1.39	2.8	1.2	1.4	1.7	1.7	0.2	0.55	1.65	inlet control	-1.5	0
2	2	1	2.73	4	0.25	0.3	2.65	1.90	3.8	2.6	0.9	1.5	2.6	0.2	0.16	2.16	inlet control	1.2	0
2	2	1	4.31	4	0.25	0.3	2.85	1.40	2.8	1.3	1.2	1.6	1.6	0.2	0.38	1.40	inlet control	-0.1	0
2	2	1	3.88	4	0.25	0.3	3.15	1.48	3.0	1.2	1.2	1.6	1.6	0.2	0.34	1.64	inlet control	-0.2	0
3	4	2	34.72	13.6	0.32	0.3	6.12	1.01	3.4	2.6	5.0	4.2	4.2	0.2	2.46	6.04	outlet control	-0.1	0
4	2.5	3	22.53	9.5	0.25	0.3	5.45	1.02	3.9	2.1	3.7	3.8	3.8	0.2	2.23	5.39	outlet control	-0.1	0
4	4.5	3	35.73	17.1	0.32	0.3	5.12	0.99	4.8	1.9	5.1	2.5	2.5	0.2	2.23	4.29	inlet control	-0.8	0
1	2	1	5.85	2	0.25	0.3	2.75	0.90	1.9	1.1	1.0	1.0	1.1	0.2	2.23	2.73	outlet control	0.0	0

The table shows the flow is not over topping so that recommended culvert for this study are acceptable.

5. CONCLUSION & RECOMMENDATION

5.1. CONCLUSION

From the result of analysis and field investigation the following specific conclusions were drawn

- Under estimated drainage area resulted in underestimated peak discharge estimation.
- Variation in length of main channel resulted variation in time of concentration
- The curve number is not defined considering the future land use and land cover change in the area, the area is highly urbanizing resulting more paved surfaces.
- Due to the above mentioned causes peak discharge was under estimated which in turn resulted in underestimating opening requirement of structure to pass the coming flood.
- Many structures on the road are clogged with wastes which increase roughness of structure and thereby reducing effective carrying capacity of structure.
- There is also a difference in design and constructed on the ground.
- Among the structures on the road 35% are not considered, 11% not consistent with the design and 50% of structures are underestimated in the original design of the road

In general the cause of flooding at *Lebu Round about to Jemo and Mikael to sebeta* is found to be Hydrologic, Hydraulic, design vs. construction mismatch and Maintenance delay.

5.2. RECOMMENDATION

- Catchment area delineation and hydrologic parameter extraction with automated computer system like GIS together with its tools will give a better output.
- Future land use and land cover change must be forecasted properly while determining Curve Number and Runoff Coefficient.
- Frequent maintenance will keep the carrying capacity of structure constant and there by effectively pass the design flood.
- Construction of structure must be consistent with design.
- Ministry of Water Irrigation and Energy of Ethiopia should update the land use land cover data. The land use and land cover map for this specific area is found to be cultivated and managed area with Agricultural land class from the data obtained, whereas the study area is more residential and land cover is also changed.
- Addis Ababa City Road Authority should check consistency of Design Vs. construction.

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7. ANNEX

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Photo 7-1.Road flooding around Megenagna.

Photo 7-2.Road flooding around Gurd Shola

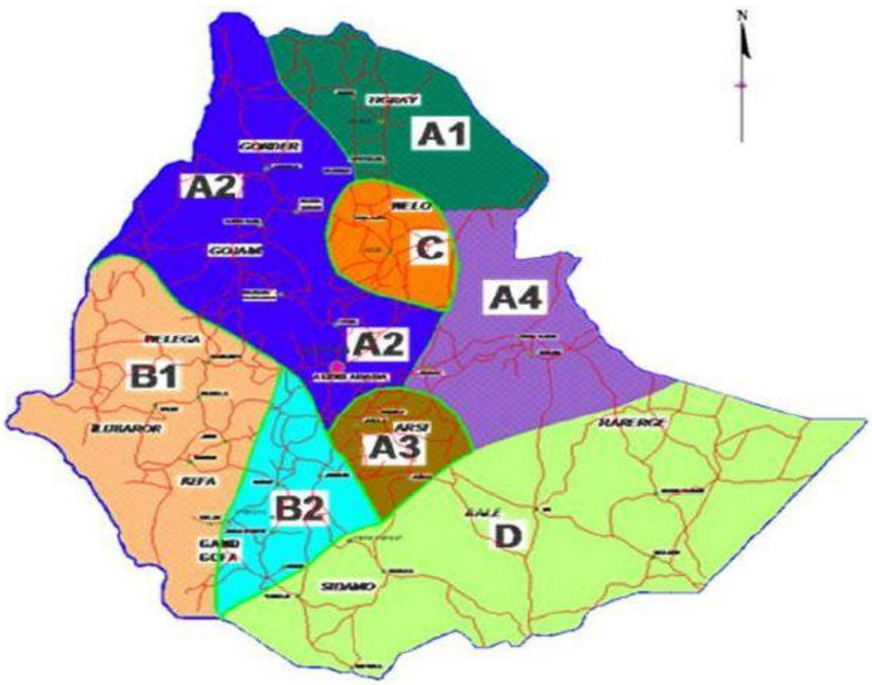
Annex 1. Photos to illustrate road flooding in different parts of Addis Ababa.

Photo 7-3. Existing situation of the road

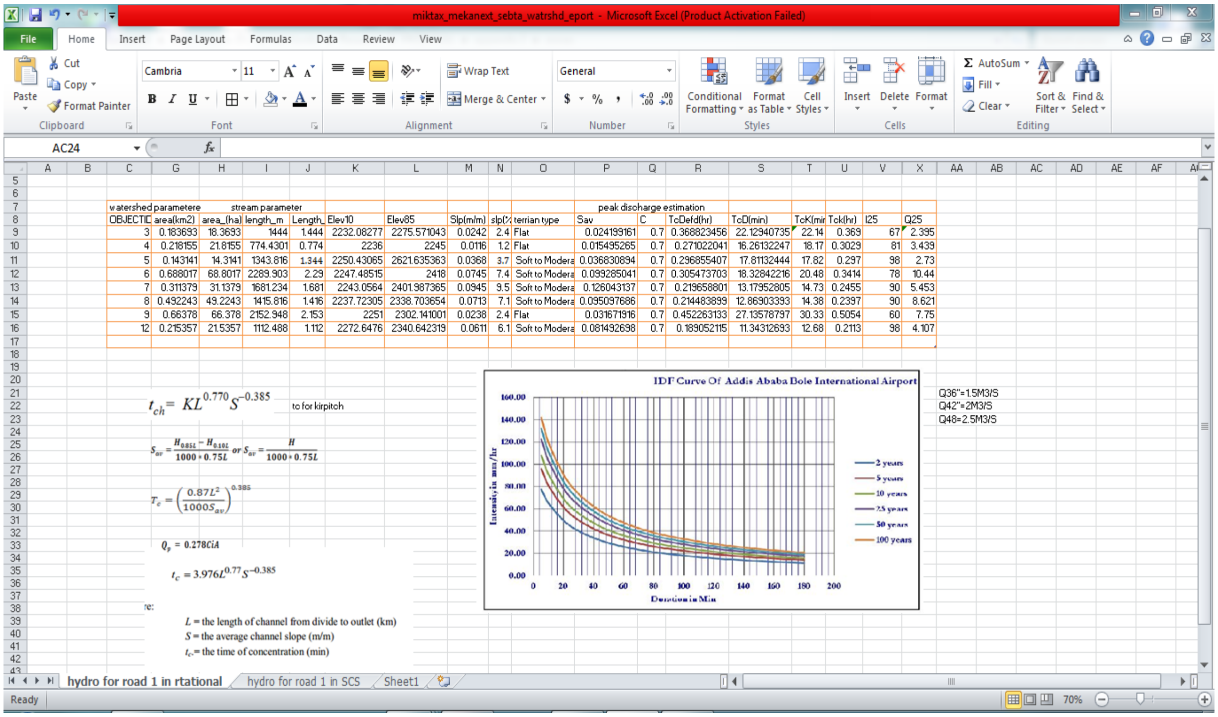
Photo 7-4. Existing situation of structures

Annex 2. The condition of pavement, the status of existing structures, the area affected by the flood and the reconstructed structure.

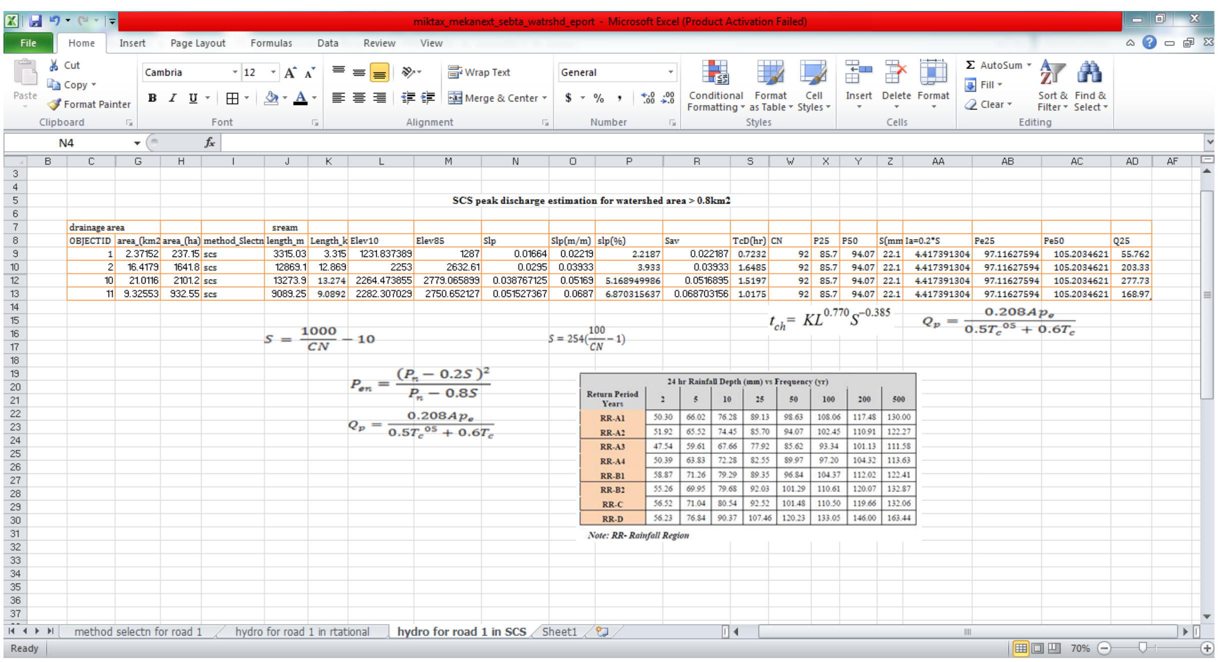
8. APPENDIX



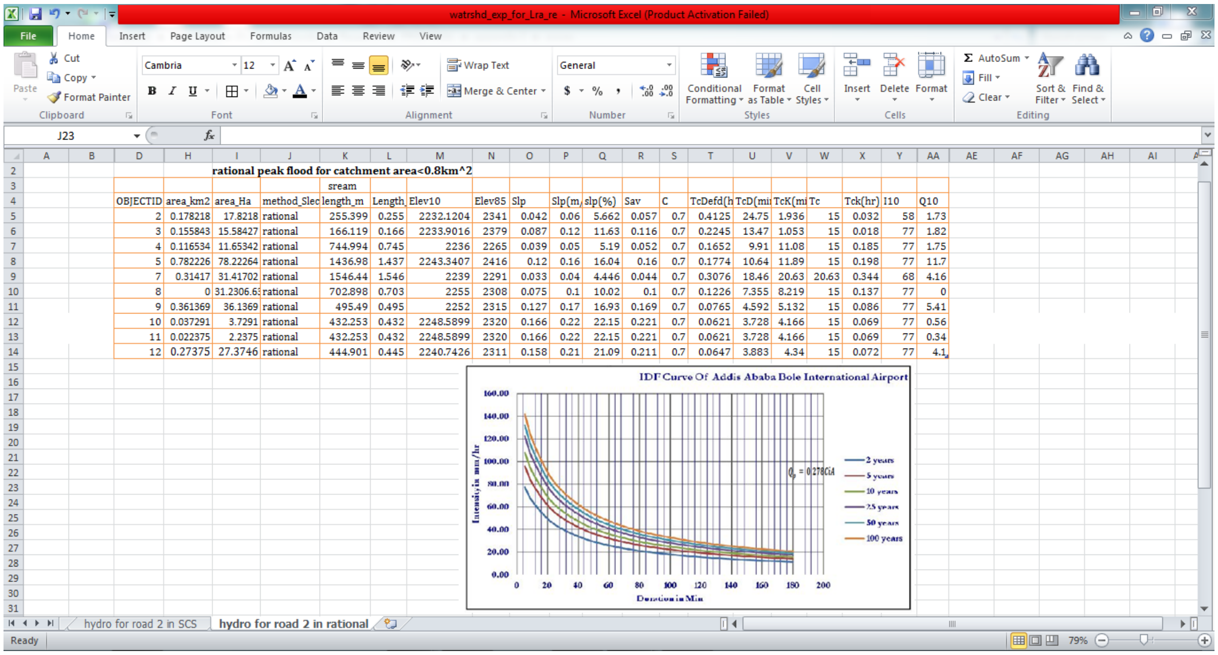
Appendix 1**.** Rain fall region of Ethiopia



Appendix 2. Rational peak discharge analysis result for road section 1



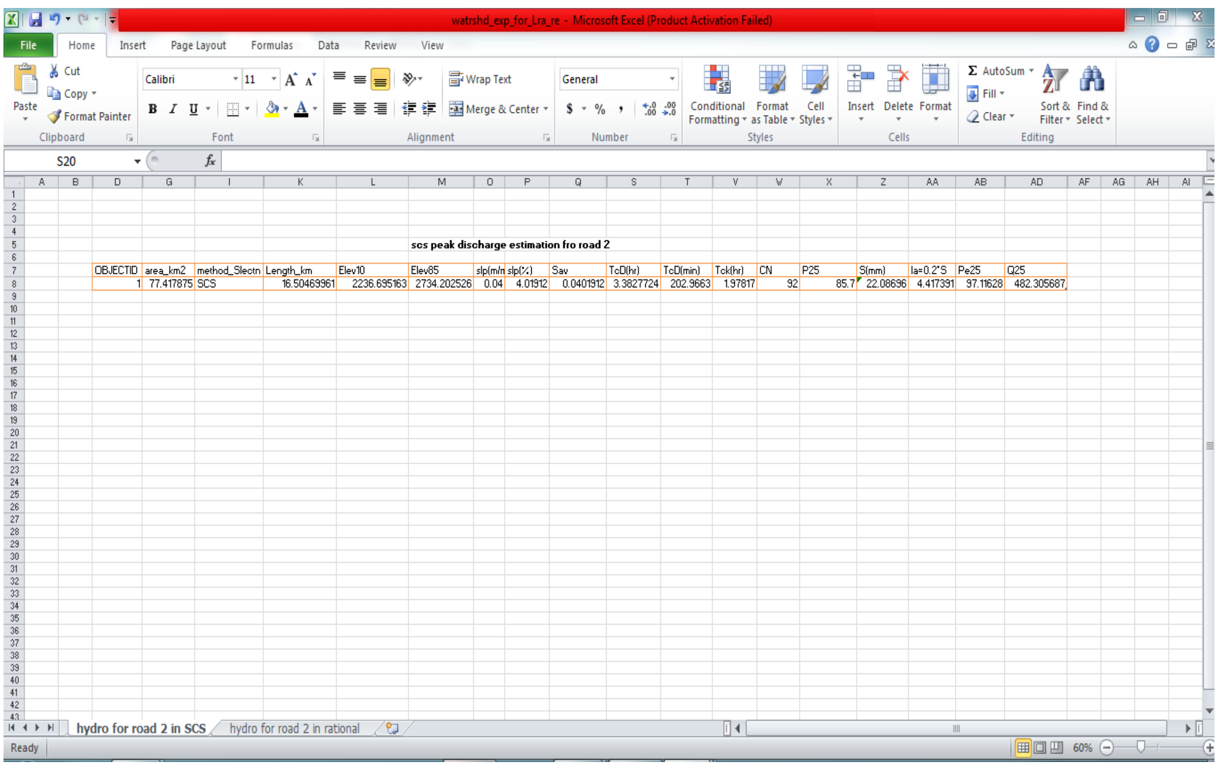
Appendix 3. SCS peak discharge analysis for road section 1



rational peak flood for catchment area<0.8km^2

OBJECTID	area_km2	area_Ha	method_Slec	sream length_m	Length_	Elev10	Elev85	Slp	Slp(m/	slp(%)	Sav	C	TcDefd(h	TcD(mi	TcK(mi	Tc	Tck(hr)	I10	Q10
2	0.178218	17.8218	rational	255.399	0.255	2232.1204	2341	0.042	0.06	5.662	0.057	0.7	0.4125	24.75	1.936	15	0.032	58	1.73
3	0.155843	15.58427	rational	166.119	0.166	2233.9016	2379	0.087	0.12	11.63	0.116	0.7	0.2245	13.47	1.053	15	0.018	77	1.82
4	0.116534	11.65342	rational	744.994	0.745	2236	2265	0.039	0.05	5.19	0.052	0.7	0.1652	9.91	11.08	15	0.185	77	1.75
5	0.782226	78.22264	rational	1436.98	1.437	2243.3407	2416	0.12	0.16	16.04	0.16	0.7	0.1774	10.64	11.89	15	0.198	77	11.7
7	0.31417	31.41702	rational	1546.44	1.546	2239	2291	0.033	0.04	4.446	0.044	0.7	0.3076	18.46	20.63	20.63	0.344	68	4.16
8	0	31.2306.6:	rational	702.898	0.703	2255	2308	0.075	0.1	10.02	0.1	0.7	0.1226	7.355	8.219	15	0.137	77	0
9	0.361369	36.1369	rational	495.49	0.495	2252	2315	0.127	0.17	16.93	0.169	0.7	0.0765	4.592	5.132	15	0.086	77	5.41
10	0.037291	3.7291	rational	432.253	0.432	2248.5899	2320	0.166	0.22	22.15	0.221	0.7	0.0621	3.728	4.166	15	0.069	77	0.56
11	0.022375	2.2375	rational	432.253	0.432	2248.5899	2320	0.166	0.22	22.15	0.221	0.7	0.0621	3.728	4.166	15	0.069	77	0.34
12	0.27375	27.3746	rational	444.901	0.445	2240.7426	2311	0.158	0.21	21.09	0.211	0.7	0.0647	3.883	4.34	15	0.072	77	4.1

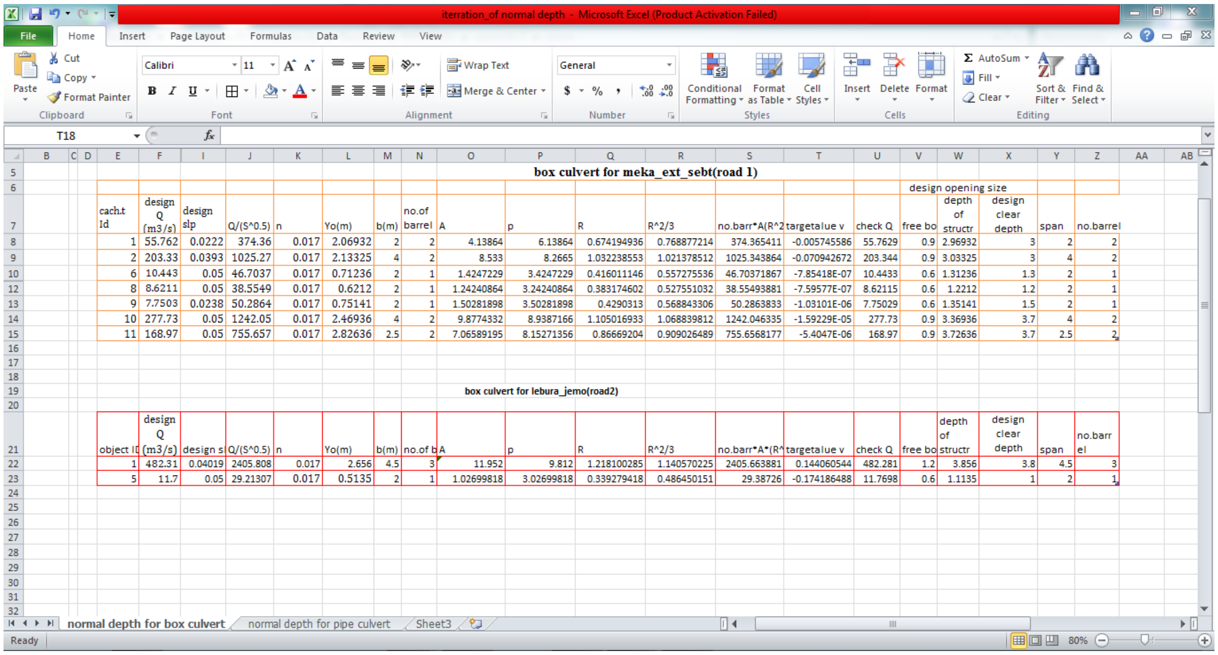
Appendix 4. Rational peak discharge analysis for road section 2



scs peak discharge estimation fro road 2

OBJECTID	area_km2	method_Slectn	Length_km	Elev10	Elev85	slp(m/n	slp(%)	Sav	TcD(hr)	TcD(min)	Tck(hr)	CN	P25	S(mm)	Ia=0.2*S	Pe25	Q25
1	77.417875	SCS	16.50469961	2236.695163	2734.202526	0.04	4.01912	0.0401912	3.3827724	202.9663	1.97817	92	85.7	22.08696	4.417391	97.11628	482.305687

Appendix 5. SCS peak discharge analysis for road section 2



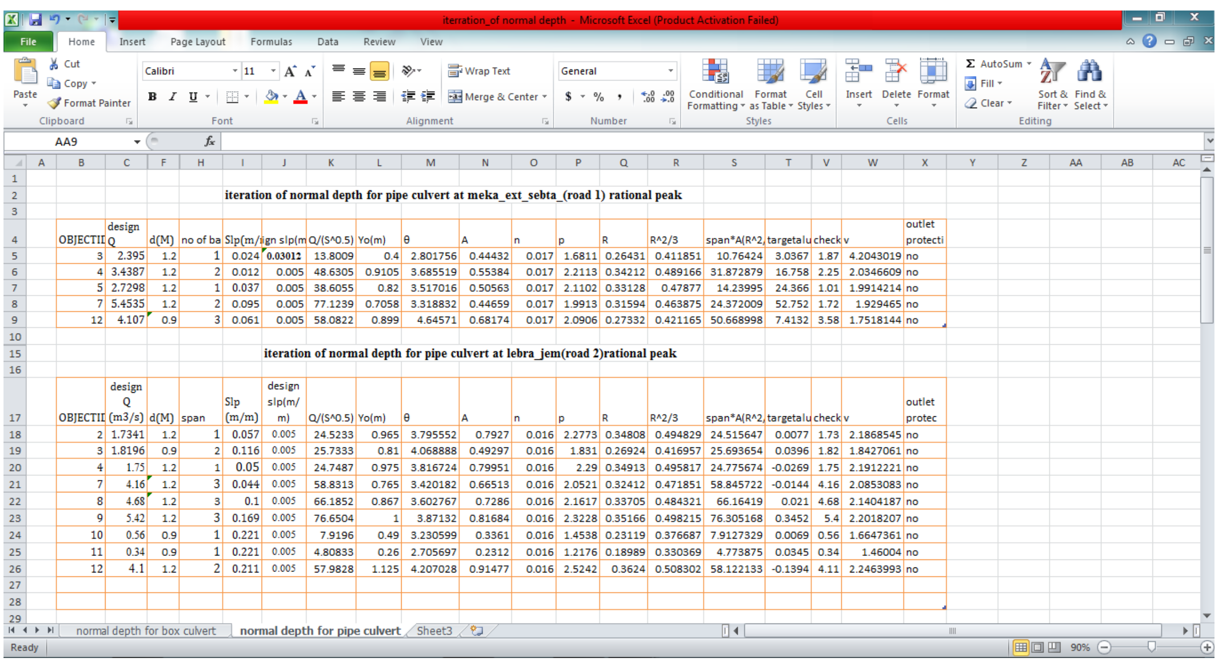
box culvert for meka_ext_sebt(road 1)

cach.t Id	design Q (m3/s)	design slp	Q/(S^0.5)	n	Yo(m)	b(m)	no.of barrel	A	p	R	R^2/3	no.barr*A(R^2	targetalue v	check Q	free bo	depth of structr	design clear depth	span	no.barrel
															design opening size				
1	55.762	0.0222	374.36	0.017	2.06932	2	2	4.13864	6.13864	0.674194936	0.768877214	374.365411	-0.005745586	55.7629	0.9	2.96932	3	2	2
2	203.33	0.0393	1025.27	0.017	2.13325	4	2	8.533	8.2665	1.032238553	1.021378512	1025.343864	-0.070942672	203.344	0.9	3.03325	3	4	2
6	10.443	0.05	46.7037	0.017	0.71236	2	1	1.4247229	3.4247229	0.416011146	0.557275536	46.70371867	-7.85418E-07	10.4433	0.6	1.31236	1.3	2	1
8	8.6211	0.05	38.5549	0.017	0.6212	2	1	1.24240864	3.24240864	0.383174602	0.527551032	38.55493881	-7.59577E-07	8.62115	0.6	1.2212	1.2	2	1
9	7.7503	0.0238	50.2864	0.017	0.75141	2	1	1.50281898	3.50281898	0.4290313	0.568843306	50.2863833	-1.03101E-06	7.75029	0.6	1.35141	1.5	2	1
10	277.73	0.05	1242.05	0.017	2.46936	4	2	9.8774332	8.9387166	1.105016933	1.068839812	1242.046335	-1.59229E-05	277.73	0.9	3.36936	3.7	4	2
11	168.97	0.05	755.657	0.017	2.82636	2.5	2	7.06589195	8.15271356	0.86669204	0.909026489	755.6568177	-5.4047E-06	168.97	0.9	3.72636	3.7	2.5	2

box culvert for lebura_jemo(road2)

object ID	design Q (m3/s)	design s	Q/(S^0.5)	n	Yo(m)	b(m)	no.of b	A	p	R	R^2/3	no.barr*A*(R^	targetalue v	check Q	free bo	depth of structr	design clear depth	span	no.barrel
1	482.31	0.04019	2405.808	0.017	2.656	4.5	3	11.952	9.812	1.218100285	1.140570225	2405.663881	0.144060544	482.281	1.2	3.856	3.8	4.5	3
5	11.7	0.05	29.21307	0.017	0.5135	2	1	1.02699818	3.02699818	0.339279418	0.486450151	29.38726	-0.174186488	11.7698	0.6	1.1135	1	2	1

Appendix 6. Hydraulic analysis result of box culvert for road section 1 and 2.



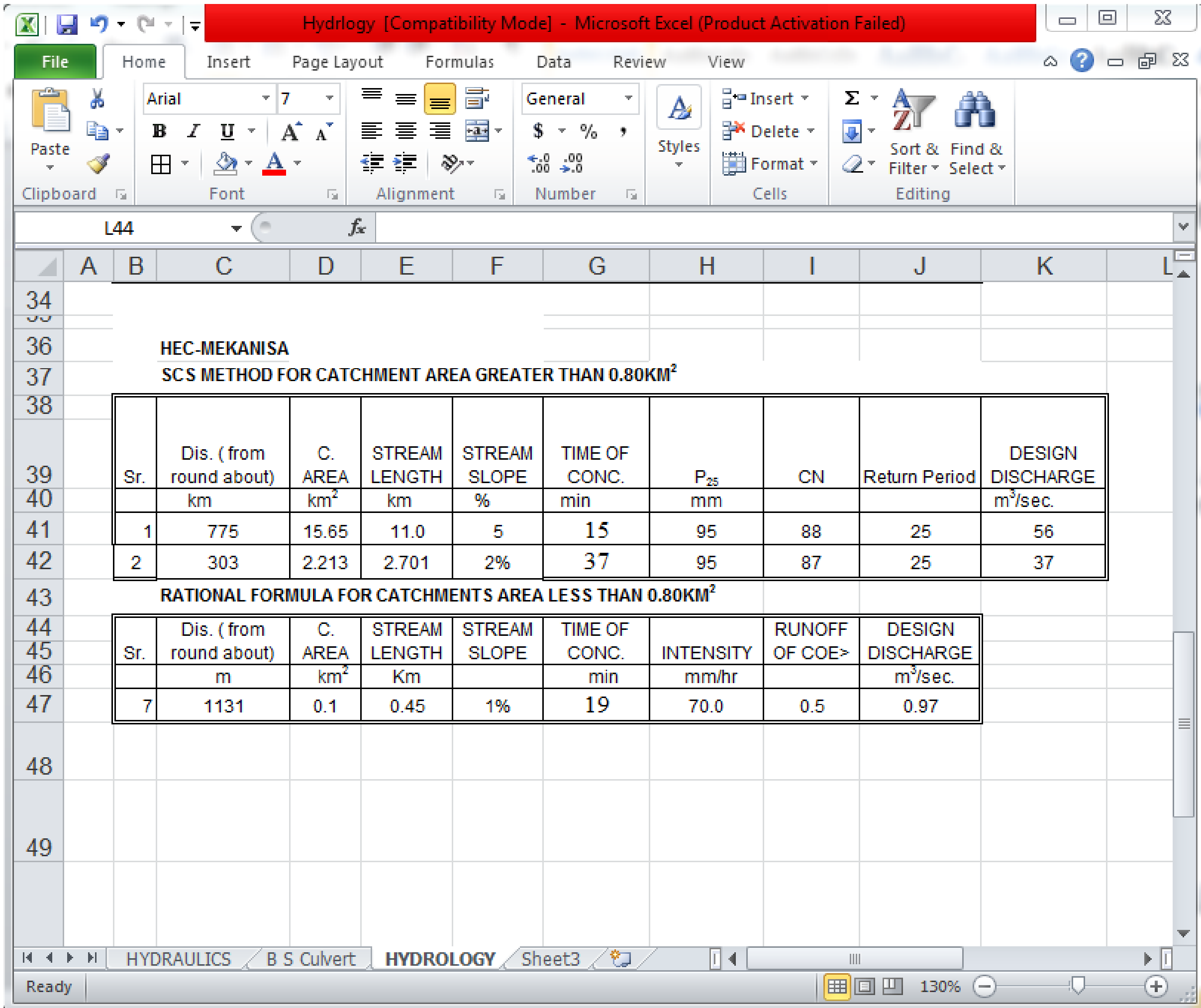
iteration of normal depth for pipe culvert at meka_ext_sebta_(road 1) rational peak

OBJECTII	design Q	d(M)	no of ba	Slp(m/	ign slp(m	Q/(S^0.5)	Yo(m)	θ	A	n	p	R	R^2/3	span*A(R^2,	targetalu	check	v	outlet protecti
3	2.395	1.2	1	0.024	0.03012	13.8009	0.4	2.801756	0.44432	0.017	1.6811	0.26431	0.411851	10.76424	3.0367	1.87	4.2043019	no
4	3.4387	1.2	2	0.012	0.005	48.6305	0.9105	3.685519	0.55384	0.017	2.2113	0.34212	0.489166	31.872879	16.758	2.25	2.0346609	no
5	2.7298	1.2	1	0.037	0.005	38.6055	0.82	3.517016	0.50563	0.017	2.1102	0.33128	0.47877	14.23995	24.366	1.01	1.9914214	no
7	5.4535	1.2	2	0.095	0.005	77.1239	0.7058	3.318832	0.44659	0.017	1.9913	0.31594	0.463875	24.372009	52.752	1.72	1.929465	no
12	4.107	0.9	3	0.061	0.005	58.0822	0.899	4.64571	0.68174	0.017	2.0906	0.27332	0.421165	50.668998	7.4132	3.58	1.7518144	no

iteration of normal depth for pipe culvert at lebra_jem(road 2)rational peak

OBJECTII	design Q (m3/s)	d(M)	span	Slp (m/m)	design slp(m/m)	Q/(S^0.5)	Yo(m)	θ	A	n	p	R	R^2/3	span*A(R^2,	targetalu	check	v	outlet protec
2	1.7341	1.2	1	0.057	0.005	24.5233	0.965	3.795552	0.7927	0.016	2.2773	0.34808	0.494829	24.515647	0.0077	1.73	2.1868545	no
3	1.8196	0.9	2	0.116	0.005	25.7333	0.81	4.068888	0.49297	0.016	1.831	0.26924	0.416957	25.693654	0.0396	1.82	1.8427061	no
4	1.75	1.2	1	0.05	0.005	24.7487	0.975	3.816724	0.79951	0.016	2.29	0.34913	0.495817	24.775674	-0.0269	1.75	2.1912221	no
7	4.16	1.2	3	0.044	0.005	58.8313	0.765	3.420182	0.66513	0.016	2.0521	0.32412	0.471851	58.845722	-0.0144	4.16	2.0853083	no
8	4.68	1.2	3	0.1	0.005	66.1852	0.867	3.602767	0.7286	0.016	2.1617	0.33705	0.484321	66.16419	0.021	4.68	2.1404187	no
9	5.42	1.2	3	0.169	0.005	76.6504	1	3.87132	0.81684	0.016	2.3228	0.35166	0.498215	76.305168	0.3452	5.4	2.2018207	no
10	0.56	0.9	1	0.221	0.005	7.9196	0.49	3.230599	0.3361	0.016	1.4538	0.23119	0.376687	7.9127329	0.0069	0.56	1.6647361	no
11	0.34	0.9	1	0.221	0.005	4.80833	0.26	2.705697	0.2312	0.016	1.2176	0.18989	0.330369	4.773875	0.0345	0.34	1.46004	no
12	4.1	1.2	2	0.211	0.005	57.9828	1.125	4.207028	0.91477	0.016	2.5242	0.3624	0.508302	58.122133	-0.1394	4.11	2.2463993	no

Appendix 7. Hydraulic analysis result of pipe culvert for road section 1 and 2.



HEC-MEKANISA

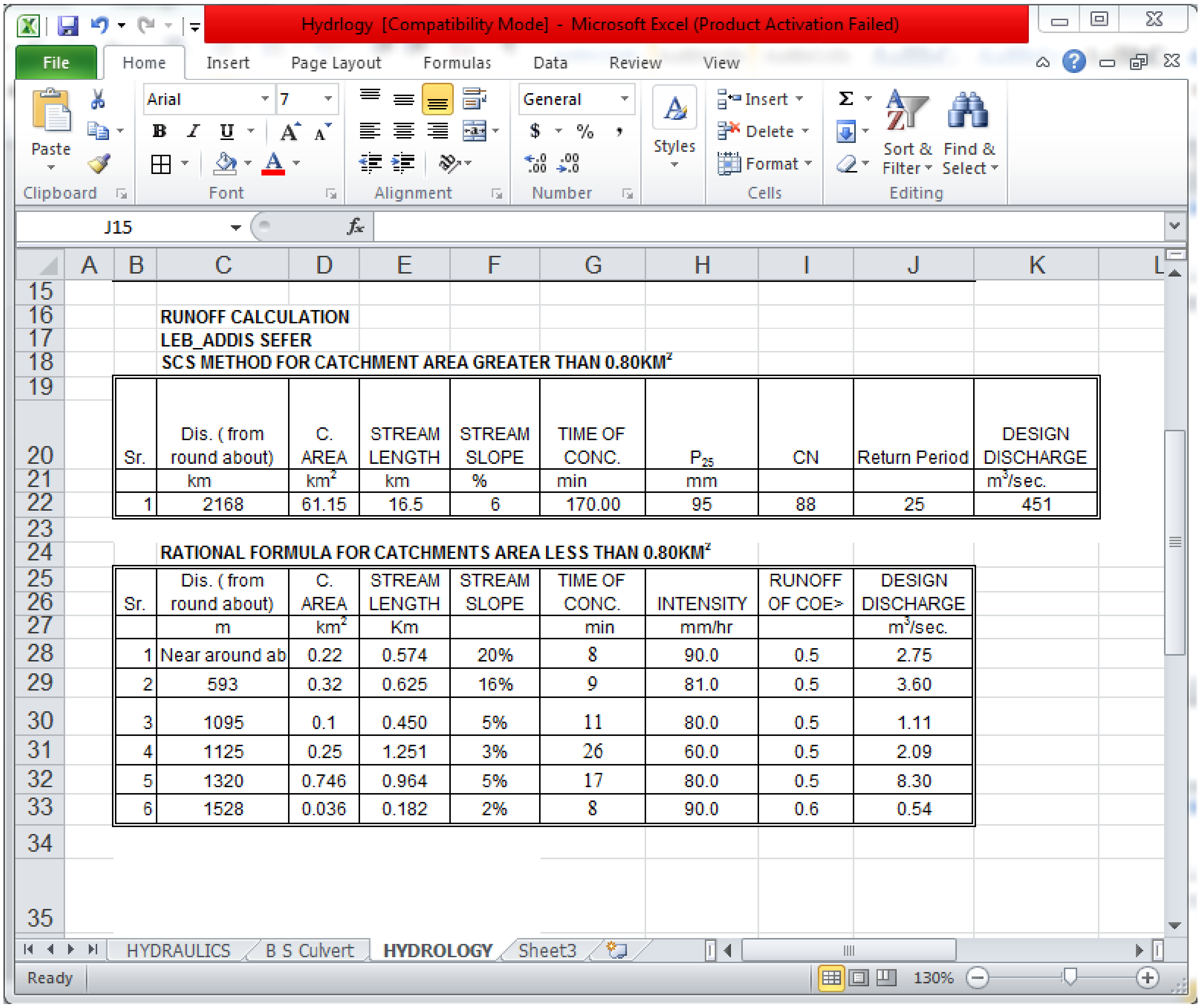
SCS METHOD FOR CATCHMENT AREA GREATER THAN $0.80KM^2$

Sr.	Dis. (from round about)	C. AREA	STREAM LENGTH	STREAM SLOPE	TIME OF CONC.	P_{25}	CN	Return Period	DESIGN DISCHARGE
	km	km^2	km	%	min	mm			$m^3/sec.$
1	775	15.65	11.0	5	15	95	88	25	56
2	303	2.213	2.701	2%	37	95	87	25	37

RATIONAL FORMULA FOR CATCHMENTS AREA LESS THAN $0.80KM^2$

Sr.	Dis. (from round about)	C. AREA	STREAM LENGTH	STREAM SLOPE	TIME OF CONC.	INTENSITY	RUNOFF OF COE>	DESIGN DISCHARGE
	m	km^2	Km		min	mm/hr		$m^3/sec.$
7	1131	0.1	0.45	1%	19	70.0	0.5	0.97

Appendix 8. Hydrology analysis and result of road section 1by HEC



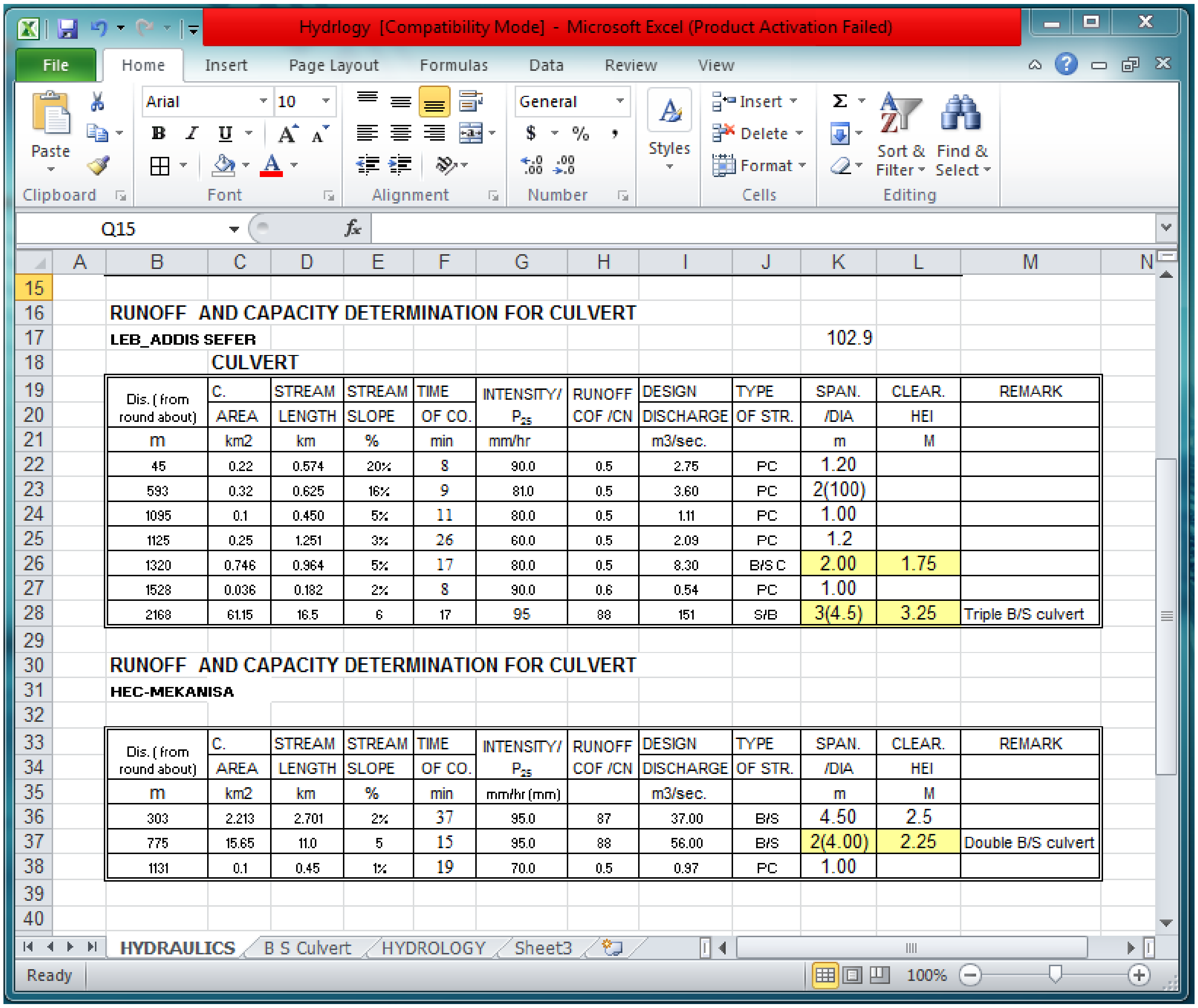
RUNOFF CALCULATION
LEB_ADDIS SEFER
SCS METHOD FOR CATCHMENT AREA GREATER THAN 0.80KM²

Sr.	Dis. (from round about)	C. AREA	STREAM LENGTH	STREAM SLOPE	TIME OF CONC.	P_{25}	CN	Return Period	DESIGN DISCHARGE
	km	km^2	km	%	min	mm			$m^3/sec.$
1	2168	61.15	16.5	6	170.00	95	88	25	451

RATIONAL FORMULA FOR CATCHMENTS AREA LESS THAN 0.80KM²

Sr.	Dis. (from round about)	C. AREA	STREAM LENGTH	STREAM SLOPE	TIME OF CONC.	INTENSITY	RUNOFF OF COE>	DESIGN DISCHARGE
	m	km^2	Km		min	mm/hr		$m^3/sec.$
1	Near around ab	0.22	0.574	20%	8	90.0	0.5	2.75
2	593	0.32	0.625	16%	9	81.0	0.5	3.60
3	1095	0.1	0.450	5%	11	80.0	0.5	1.11
4	1125	0.25	1.251	3%	26	60.0	0.5	2.09
5	1320	0.746	0.964	5%	17	80.0	0.5	8.30
6	1528	0.036	0.182	2%	8	90.0	0.6	0.54

Appendix 9. Hydrology analysis and result of road section 2 (Lebu RA to Jemo)



RUNOFF AND CAPACITY DETERMINATION FOR CULVERT

LEB_ADDIS SEFER 102.9

CULVERT

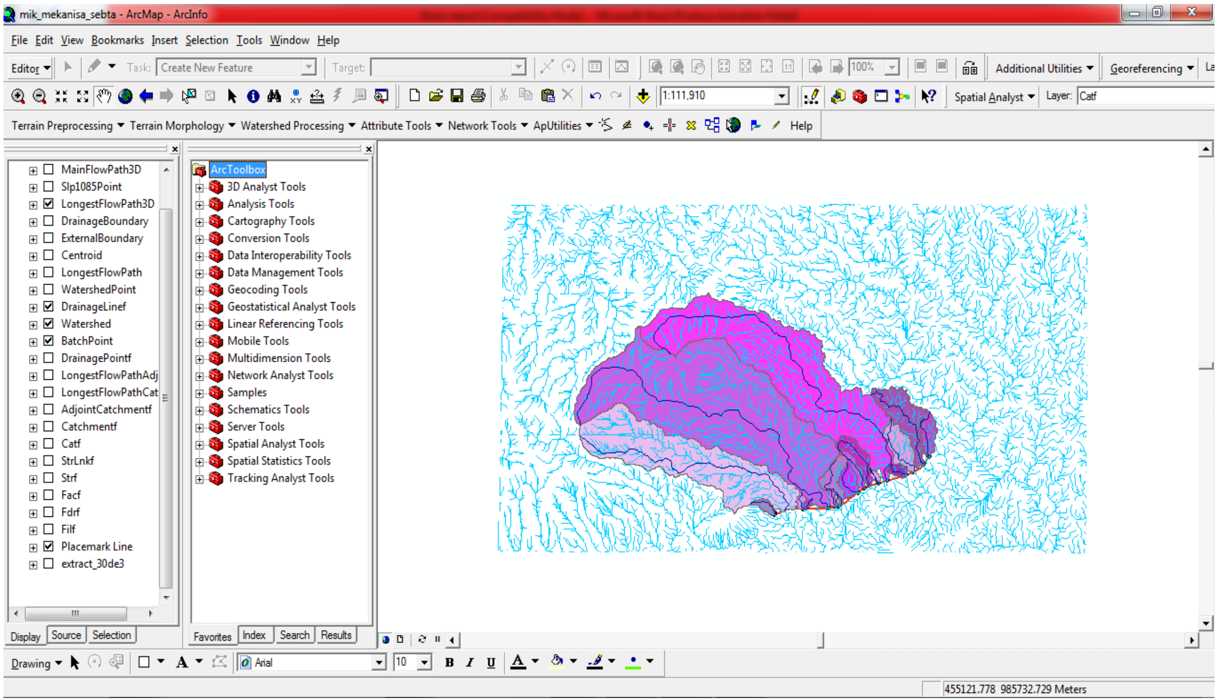
Dis. (from round about)	C. AREA	STREAM LENGTH	STREAM SLOPE	TIME OF CO.	INTENSITY/ P_{25}	RUNOFF COF /CN	DESIGN DISCHARGE	TYPE OF STR.	SPAN. /DIA	CLEAR. HEI	REMARK
m	km2	km	%	min	mm/hr		m3/sec.		m	M	
45	0.22	0.574	20%	8	90.0	0.5	2.75	PC	1.20		
593	0.32	0.625	16%	9	81.0	0.5	3.60	PC	2(100)		
1095	0.1	0.450	5%	11	80.0	0.5	1.11	PC	1.00		
1125	0.25	1.251	3%	26	60.0	0.5	2.09	PC	1.2		
1320	0.746	0.964	5%	17	80.0	0.5	8.30	B/S C	2.00	1.75	
1528	0.036	0.182	2%	8	90.0	0.6	0.54	PC	1.00		
2168	61.15	16.5	6	17	95	88	151	S/B	3(4.5)	3.25	Triple B/S culvert

RUNOFF AND CAPACITY DETERMINATION FOR CULVERT

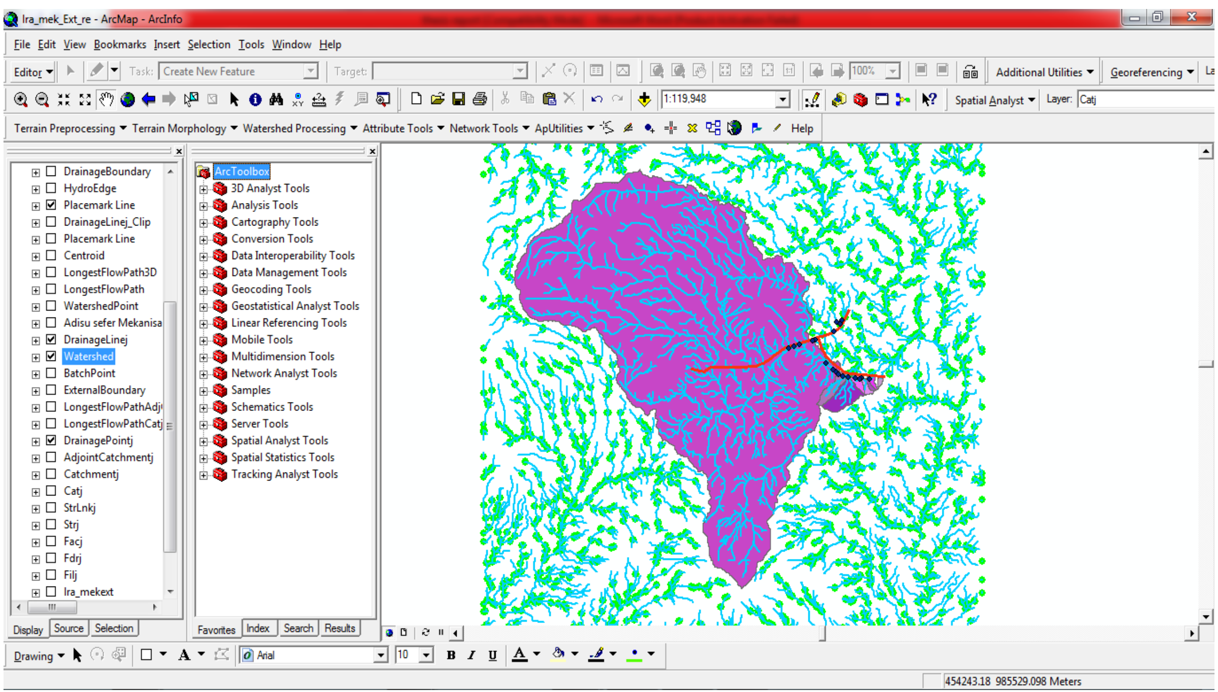
HEC-MEKANISA

Dis. (from round about)	C. AREA	STREAM LENGTH	STREAM SLOPE	TIME OF CO.	INTENSITY/ P_{25}	RUNOFF COF /CN	DESIGN DISCHARGE	TYPE OF STR.	SPAN. /DIA	CLEAR. HEI	REMARK
m	km2	km	%	min	mm/hr (mm)		m3/sec.		m	M	
303	2.213	2.701	2%	37	95.0	87	37.00	B/S	4.50	2.5	
775	15.65	11.0	5	15	95.0	88	56.00	B/S	2(4.00)	2.25	Double B/S culvert
1131	0.1	0.45	1%	19	70.0	0.5	0.97	PC	1.00		

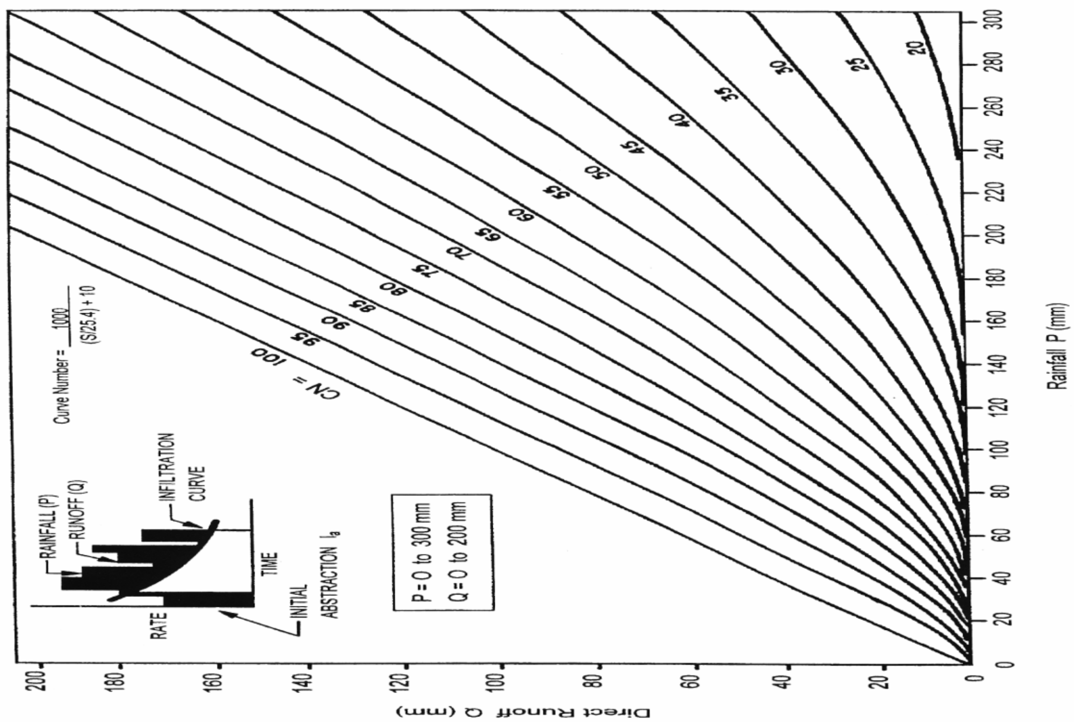
Appendix 10. Hydraulics analysis and result for road section 1 and 2 by HEC



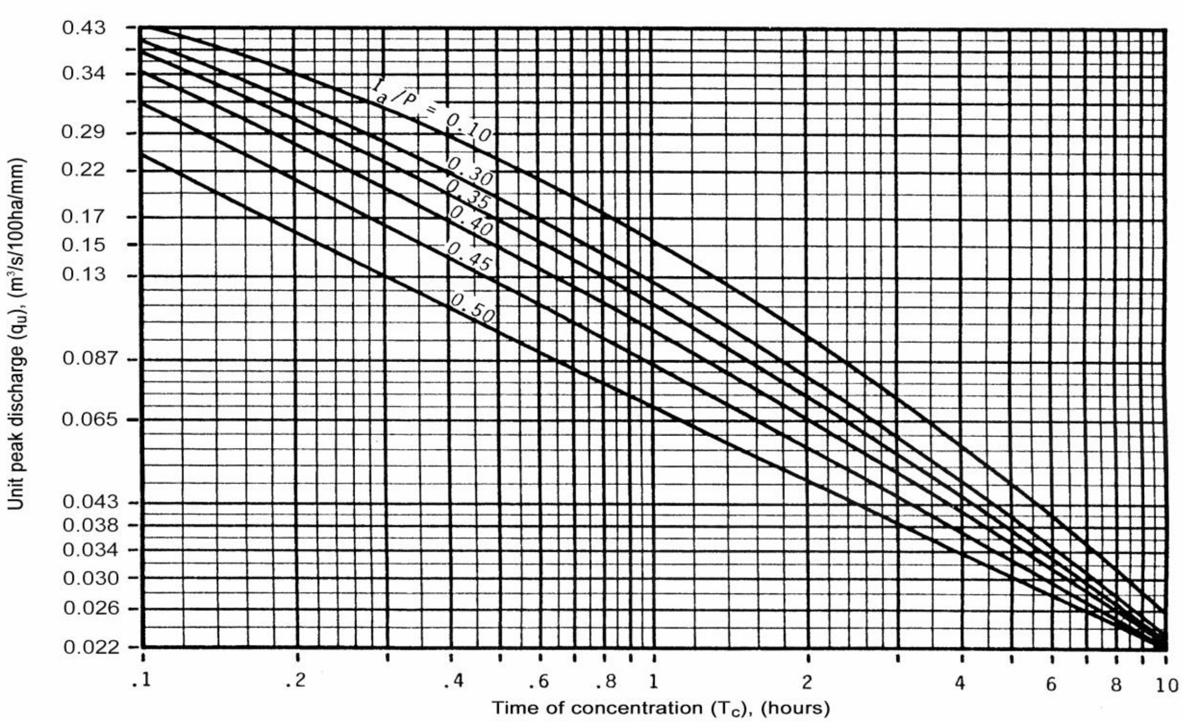
Appendix 11. Watersheds for Road section 1



Appendix 12. Watersheds for Road section 2



Appendix 13. SCS Relation between Direct Runoff, Curve Number and Precipitation



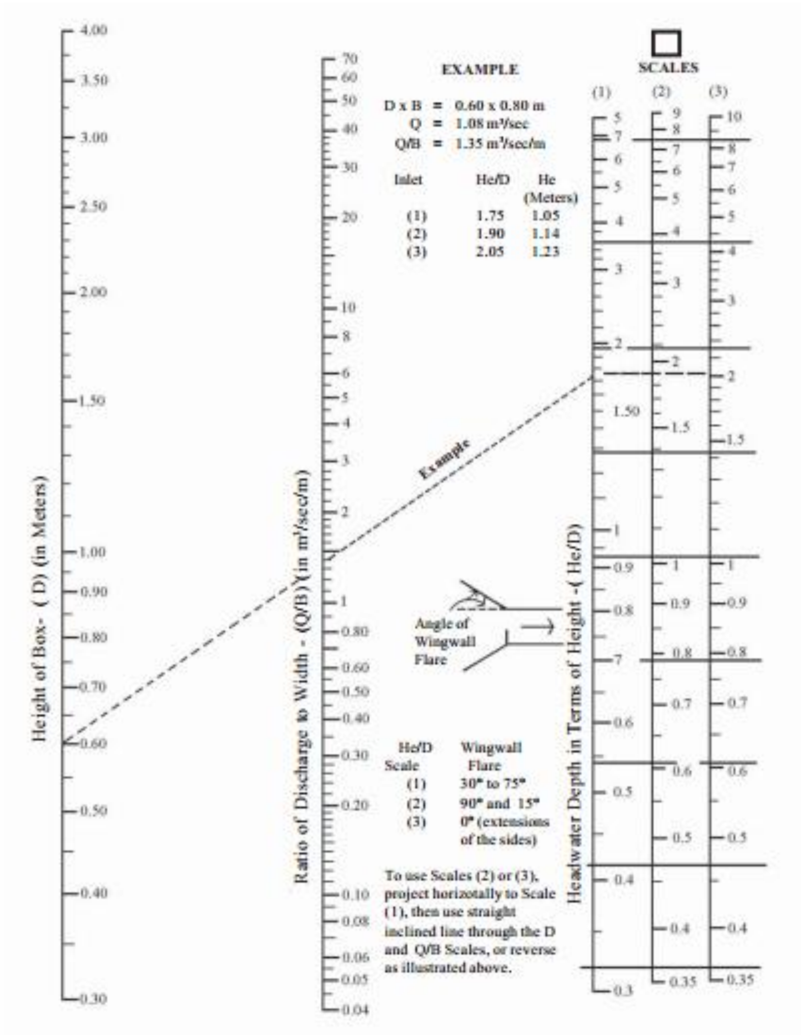
Appendix 14. Unit peak discharge, type II rain fall

Type of Drainage Area	Runoff Coefficient, C*
Business:	
Downtown areas	0.70 - 0.95
Neighborhood areas	0.50 - 0.70
Residential:	
Single-family areas	0.30 - 0.50
Multi-units, detached	0.40 - 0.60
Multi-units, attached	0.60 - 0.75
Suburban	0.25 - 0.40
Apartment dwelling areas	0.50 - 0.70
Industrial:	
Light areas	0.50 - 0.80
Heavy areas	0.60 - 0.90
Parks, cemeteries	0.10 - 0.25
Playgrounds	0.20 - 0.40
Railroad yard areas	0.20 - 0.40
Unimproved areas	0.10 - 0.30
Lawns:	
Sandy soil, flat, 2%	0.05 - 0.10
Sandy soil, average, 2 - 7%	0.10 - 0.15
Sandy soil, steep, 7%	0.15 - 0.20
Heavy soil, flat, 2%	0.13 - 0.17
Heavy soil, average, 2 - 7%	0.18 - 0.22
Heavy soil, steep, 7%	0.25 - 0.35
Streets:	
Asphaltic	0.70 - 0.95
Concrete	0.80 - 0.95
Brick	0.70 - 0.85
Drives and walks	0.75 - 0.85
Roofs	0.75 - 0.95
*Higher values are usually appropriate for steeply sloped areas and longer return periods because infiltration and other losses have a proportionally smaller effect on runoff in these cases.	

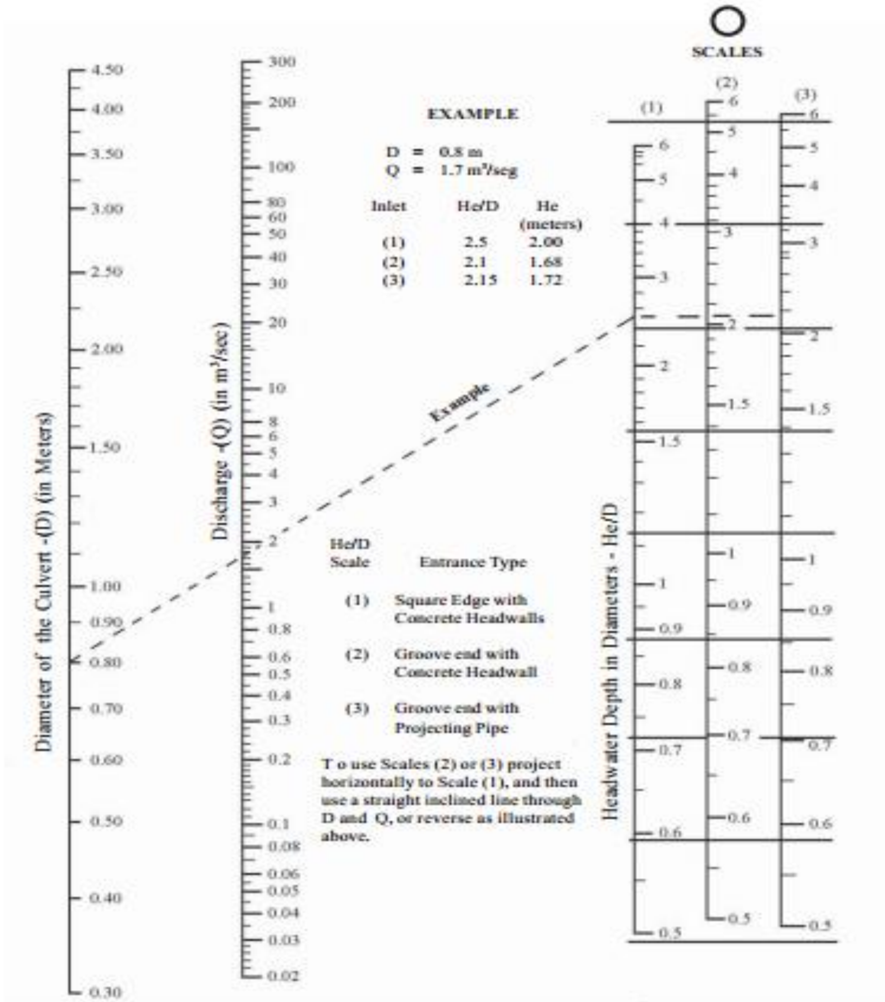
Appendix 15. Runoff Coefficients for Rational Formula (FHWA, DDM, HEC22)

Rainfall Type	Ia/P	C_0	C_1	C_2
I	0.1	2.3055	-0.5143	-0.1175
	0.2	2.23537	-0.5039	-0.0893
	0.25	2.18219	-0.4849	-0.0659
	0.3	2.10624	-0.4570	-0.0284
	0.35	2.00303	-0.4077	0.01983
	0.4	1.87733	-0.3227	0.05754
	0.45	1.76312	-0.1564	0.00453
	0.5	1.67889	-0.0693	0.00000
IA	0.1	2.03250	-0.3158	-0.1375
	0.2	1.91978	-0.2822	-0.0702
	0.25	1.83842	-0.2554	-0.0260
	0.3	1.72657	-0.1983	0.02633
	0.5	1.63417	-0.0910	0.0000
II	**0.1**	**2.55323**	**-0.6151**	**-0.1640**
	0.3	**2.46532**	**-0.6226**	**-0.1166**
	0.35	**2.41896**	**-0.6159**	**-0.0882**
	0.4	**2.36409**	**-0.5986**	**-0.0562**
	0.45	**2.29238**	**-0.5701**	**-0.0228**
	0.5	**2.20282**	**-0.5160**	**-0.0126**
III	0.1	2.47317	-0.5185	-0.1708
	0.3	2.39628	-0.512	-0.1325
	0.35	2.35477	-0.4974	-0.1199
	0.4	2.30726	-0.4654	-0.1109
	0.45	2.24876	-0.4131	-0.1151
	0.5	2.17772	-0.3680	-0.0953

Appendix 16. Regression coefficients for SCS unit peak discharge method.



Appendix 17 Headwater depth and capacity for concrete box culverts with inlet control. (FHWA, HDS5, 1998)



Appendix 18 Headwater depth and capacity for concrete pipe culverts with inlet control.

(FHWA, HDS 5, 1998)